Section 3 — Pavement Design (New Construction)

Section 3A

Evaluation of Site Conditions

Introduction

The purpose of this section is to identify UDOT standards, practices and references; and to provide guidelines and work aids for site evaluations for the design of new pavements. This section applies to site investigations for new construction performed for pavement thickness design, including the development of performance related earthwork or pavement drainage designs. This category of site investigations primarily obtains information regarding soil support conditions and drainage requirements.

Site investigations for geometric roadway locations, excavation or embankment design, slope stability analysis, structure design, construction material sources or pavement rehabilitation are not part of this section.

Scope of Site Evaluations

Site evaluations for new pavement design and construction are performed following a defined testing plan that helps identify the necessary conditions to ensure the required information is obtained in the field.

Required Design Information

The following list includes information that is generally useful or necessary for pavement design. However, site investigations for pavement design are not intended to identify all conditions that could affect construction. Specific projects may require only some of the information listed below or may require additional information. The elimination or addition of information should be determined during the initial consultant design-concept meeting.

Site Investigation Data

1 soil types exposed at the ground surface.

2 soil conditions within a depth of 3 to 5 feet below the pavement subgrade including:

- a. soil classification units
- b. in place soil moisture content and density
- c. the occurrence of bedrock or boulders
- d. the occurrence of swelling soils
- e. soil plastic and liquid limits
- f. moisture-density compaction curves
- g. the occurrence of moisture induced collapsing soils
- h. potential sources of settlement
- i. laboratory and field CBR values

3 the depth to groundwater below the pavement subgrade
4. subgrade support variability
5. the approximate vertical distance of the pavement surface above or below the adjacent ground surface
6. soft or weak soils that will not support or will limit the size of earthwork equipment
7. vegetation, debris and other deleterious material that may affect pavement support.
8. a hazard rating for frost damage
9. water hazards
10. performance of nearby pavements
11. design CBR for road sections

Site Evaluation/Testing Plans

Site evaluation/testing plans shall be developed by or with the concurrence of the Regional Pavement Management Engineer. Coordination with UDOT's geotechnical section shall be undertaken when the project includes soils investigations for purposes other than pavement design. These plans shall include exploration and sampling strategy, preliminary boring or test pit locations, and testing criteria. The information listed in the preceding section shall be used as a check list for developing plans.

Field Investigations

Field investigations include the planning, locating, drilling and collection of borings and samples for laboratory testing. Information regarding the alignment and preliminary grade should be obtained before a field investigation begins to assist in the proper location of the test holes. Soils shall be sampled to determine both the vertical and horizontal boundaries of each significant soil type affecting the pavement support.

Objectives for sampling include 1. classifying soils, 2. obtaining material for laboratory testing, 3. characterizing material pavement support characteristics, and 4. delineating boundaries of soils with varying pavement support values or verifying or documenting uniformity. The number and size of the samples shall be adequate for determining soil classification, moisture-density curves, CBR values and other laboratory testing appropriate for a specific project. Before the field investigation is completed, a preliminary laboratory testing plan shall be developed and integrated into the field testing plan to assure that the number and size of samples is adequate. Contingency samples shall be taken to accommodate unexpected variations of laboratory test results.

Boring/Sample Locations

Location of the borings and samples should be determined based on the centerline location of the planned road, the planned width of the road and the expected soil conditions for the area. Spacing of the test holes will be controlled by the type and profile of the soil at each location. For long road sections, UDOT practice includes using 200 feet as a starting interval for exploration locations and varying this interval up to a maximum of 1000 feet for uniform conditions. If the soil types significantly change between test holes, intermediate locations shall be investigated.

The determination of the number and location of samples and/or borings shall consider the reliability of the pavement design and the cost-effectiveness of the investigation. Borings, samples, and other explorations shall be located so that the sites can be found during construction. The locations shall be referenced to 1. a construction station, 2. road centerline and 3. elevations or road grades where possible.

Sampling Techniques

There are two basic sampling techniques that can be applied to the investigations, systematic or representative. UDOT's basic practice is to use systematic sampling.

Systematic Sampling
Systematic sampling is the most common UDOT practice. It is done at uniform horizontal or vertical intervals. Intermediate locations are sampled when varying conditions are encountered. All the soil samples are classified either visually or in the laboratory.

Representative Sampling
Representative sampling and testing consists of taking samples that are believed to be representative of the typical or conservative soil support values. This type of sampling is based primarily on personal judgment, involves a few samples, and generally is only appropriate for small projects or verifying existing data.

Laboratory Investigations

Laboratory investigations involve the performance of all required testing and classification necessary to adequately identify the soil conditions for the entire project. The testing should be performed in accordance with the appropriate AASHTO, ASTM and UDOT procedures and should be well documented.

Soil Classification

Soils are classified to group material with similar engineering properties and to identify soil types specified in construction contracts. All soil samples shall be classified according to the AASHTO Soil Classification System. This classification system is used in UDOT construction specifications. The use of the Unified Soil Classification System may be helpful in identifying soil classes with smaller ranges of CBR values than the soil classes in the AASHTO system. Figure 3A-1 in the appendix of this section shows typical CBR ranges for soil classes in both the AASHTO and Unified Classification systems for AASHTO T-180 maximum densities. Figure 3A-2 in the appendix of this section shows a comparison of the gradation and plasticity criteria used in the AASHTO and Unified Soil Classification Systems.

Selection of Design CBR

Subgrade soil CBR values shall be determined using samples compacted at optimum moisture content to 100 percent of the maximum density obtainable by the AASHTO T-99D method of compaction. CBR tests shall be performed according to AASHTO T-193 except that a standard surcharge weight of 10 pounds shall be used for soaking and the penetration test of all samples.

A CBR chosen for pavement design purposes shall have a confidence level of 90% for a normal distribution of values. If it is determined that there is an insufficient number of CBR tests, then the lowest CBR value will be used for design. This will be determined by the Regional Pavement Management Engineer. The following table indicates the number of CBR tests that will provide a 90 percent confidence level that the average test value is within plus or minus 1 unit of the average for a normal t-distribution of values.

CBR Test Range	1	2	3	4	5	6
Number of Required Tests	2	3	4	5	6	8-9

Table 3A-1. Correlation of Range vs. Number of Tests for 90% Confidence

Reliability (safety) is a specific input factor in the 1993 *AASHTO Design Procedure for Pavement Structures*. Use of the AASHTO design procedure reliability factor in the structural thickness design will account for variability in subgrade support values relative to an average or typical value.

Presentation of Data

As part of the project pavement design, the results of the site investigation should be documented and tabulated for use and review in the pavement design process. The following site information shall be included with a pavement design report:

1. A summary of the soil types and test results on site plans that show the sample or test hole locations. Figure 3A-3 in the appendix of this section presents example subsurface summaries.

2. CBR test results including load-penetration curves, moisture contents at set up/after swell/ top 1 inch after penetration, percent relative compaction, compaction method, % swell, AASHTO soil classification, liquid limit and plastic limit.

3. Any conditions which controlled the pavement design, such as soil type and subgrade moisture conditions, shall be verified during construction. If conditions are significantly different from those determined by the site evaluation, the pavement design should be reviewed and modified if necessary.

4. Climatic conditions that will require changes or alterations to the structural pavement sections. These items include such issues as frost hazards, water hazards and unstable subgrades.

Frost Hazard Identification

Frost action creates a hazard to pavement performance by causing the surface to heave and by decreasing the soil support by increasing the moisture content above the normal saturation content. The following three conditions must exist for a pavement frost hazard to exist:

1. temperatures below freezing

2. a water table above the frost line or a source of water that can saturate the soils above the frost line by infiltration or capillary action.

3. a grain size distribution that will result in frost susceptible pore sizes.

Figure 3A-4 shows typical depths to the frost line at varying locations within the State of Utah. Many of the soils above the frost line in Utah may be classified as frost susceptible according to grain size classification criteria. The most distinguishing factor for identifying pavement frost hazards is condition 2. above, water supply. For frost susceptible soils within the frost zone, the frost hazard may be rated as high or low according to the following conditions. An unknown rating may be appropriate when conditions for both high and low ratings occur and can not be resolved or when little or no information is available. The inclusion of a frost hazard rating in the site evaluation documentation verifies that an evaluation of frost action has been attempted and has not been overlooked. When the rating is unknown, a decision to include frost action mitigation measures in a design will be based more upon the unacceptableness of frost damage than the probability of occurrence.

The conditions associated with a high frost hazard potential include:

1. a water table within 10 feet of the pavement surface

2. observed frost heaves in the area

3. the U.S. Army Corps of Engineers considers inorganic soils containing more than 3 percent (by weight) or more grains finer than 0.02 mm in diameter to be frost susceptible for pavement design purposes.

4. a potential for the ponding of surface water and the occurrence of soils between the frost zone beneath the pavement and the surface water with permeabilities high enough to enable seepage to saturate soils within the frost zone beneath the pavement during the term of ponding.

The conditions associated with a low frost hazard potential include:

1. a water table greater than 20 feet below the pavement surface

2. natural moisture content in the frost zone low versus the saturation level

3. seepage barriers between the water supply and the frost zone

4. existing pavements or side walks in the vicinity with similar soil and water supply conditions and without constructed frost protection measures that have not experienced frost damage

5. pavements on embankments with surfaces more than 3 to 6 feet above the adjacent grades

Water Hazards

Water hazards are those conditions that may require the design of drainage provisions to improve pavement performance.

Conditions that may be classified as water hazards include:

1. a potential for surface water to pond in close proximity to pavement

2. a water table within a 6 foot depth of the pavement surface.

3. a spring beneath the pavement

4. Intermittent sources of water that could cause seepage beneath and within a 5 foot depth of the pavement surface.

Appendix 3-A

Standard Methods

1. AASHTO T-193 for determining the CBR (California Bearing Ratio. of laboratory-compacted soils as modified by using a standard 10 pound surcharge weight.
3. ASTM D 4318 for determining the liquid limit, plastic limit and plastic index of soils.
4. AASHTO M 145 for determining the AASHTO Classification of soils and soil-aggregate mixtures.
5. ASTM D 2487 for determining the Unified Classification of soils for engineering purposes.

Sources of Information

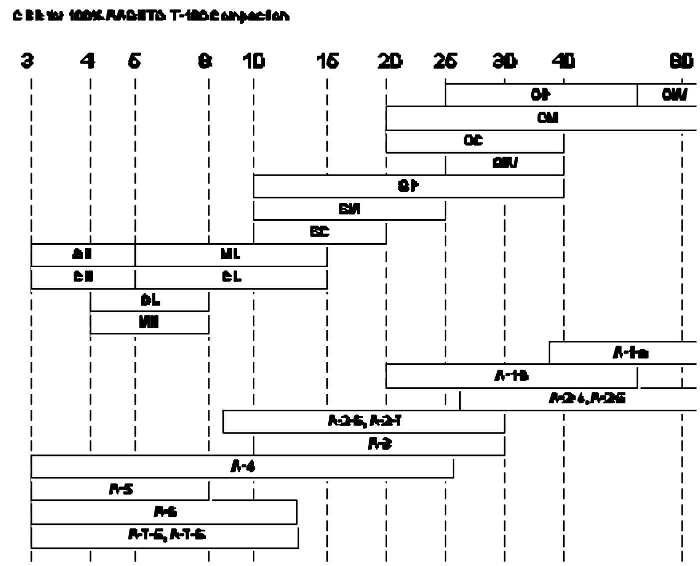
Prior to starting a field investigation, the information available for the site or locations nearby should be reviewed. Useful sources of information include:

1. geologic and topographic maps
2. existing UDOT soil surveys
3. existing pavement designs
4. as-built construction plans
5. UDOT annual pavement condition surveys
6. UDOT system and project level Falling Weight Deflectometer (FWD) investigations

Falling Weight Deflectometer (FWD) surveys are performed periodically on existing roads. Deflection readings are taken at each mile point using a Falling Weight Deflectometer (FWD) to test approximately 20% of all paved highways on the State System each year. Roads and parking areas associated with institutional buildings are not included in the surveys. The structural value of the pavement/base material (EPAVE) and subgrade (ESUB) are reported.

Consultants shall contact the Regional Pavement Management Engineers to determine which of the above UDOT information is available.

Anticipated CBR Ranges by Soil Classification



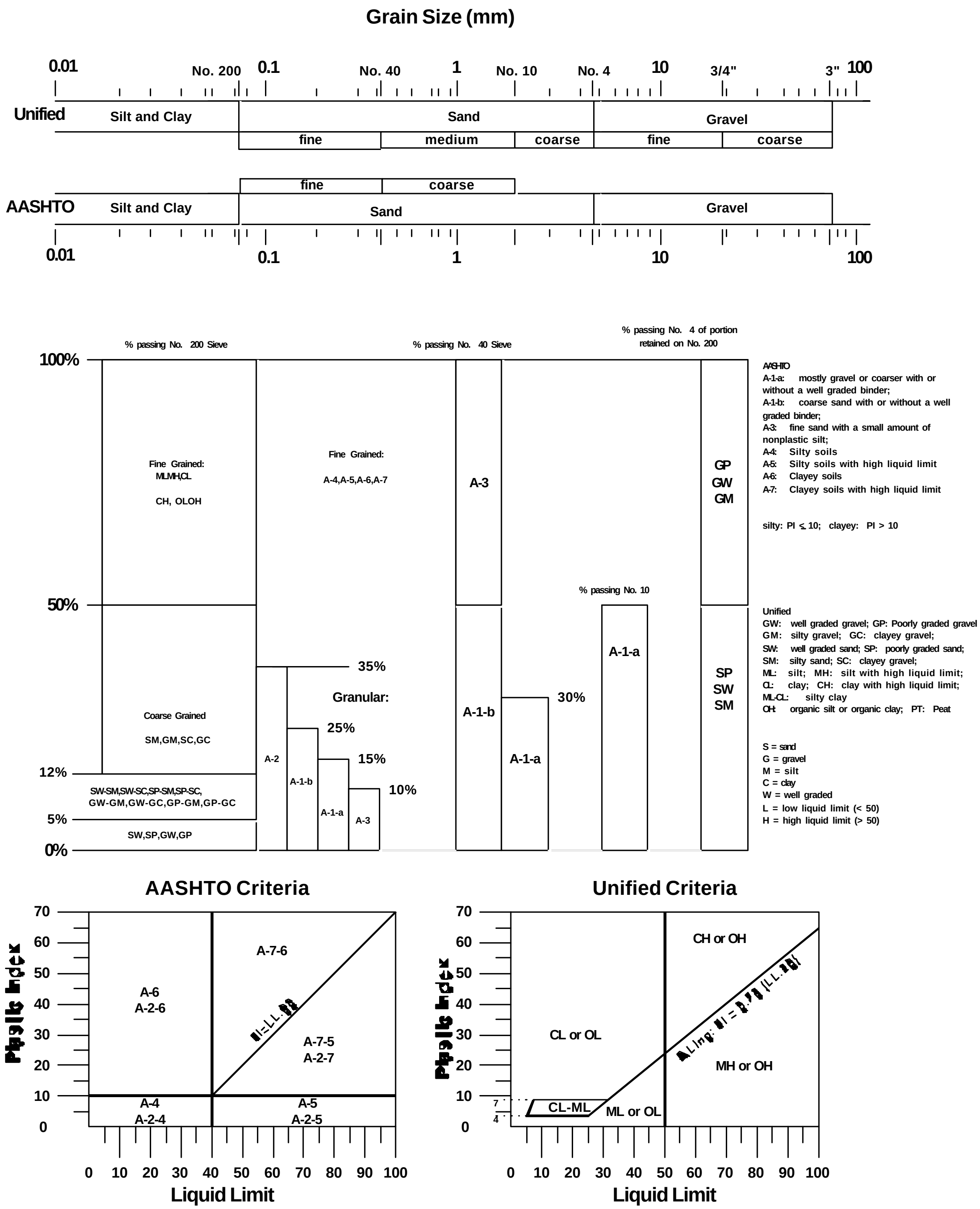
Cohesive Soils: Multiply chart CBR values by 0.5 to estimate CBR for 100% T-99 compaction level. Multiply CBR value at 100% T-99 compaction level by 0.5 to estimate CBR value at 95% T-99 compaction level.

Granular Soils: Multiply chart CBR values by 0.67 to estimate CBR for 100% T-99 compaction level. Multiply CBR value at 100% T-99 compaction level by 0.75 to estimate CBR value at 95% T-99 compaction level.

Intermediate Soils: Multiply chart CBR values by 0.6 to estimate CBR for 100% T-99 compaction level. Multiply CBR value at 100% T-99 compaction level by 0.5 to estimate CBR value at 95% T-99 compaction level.

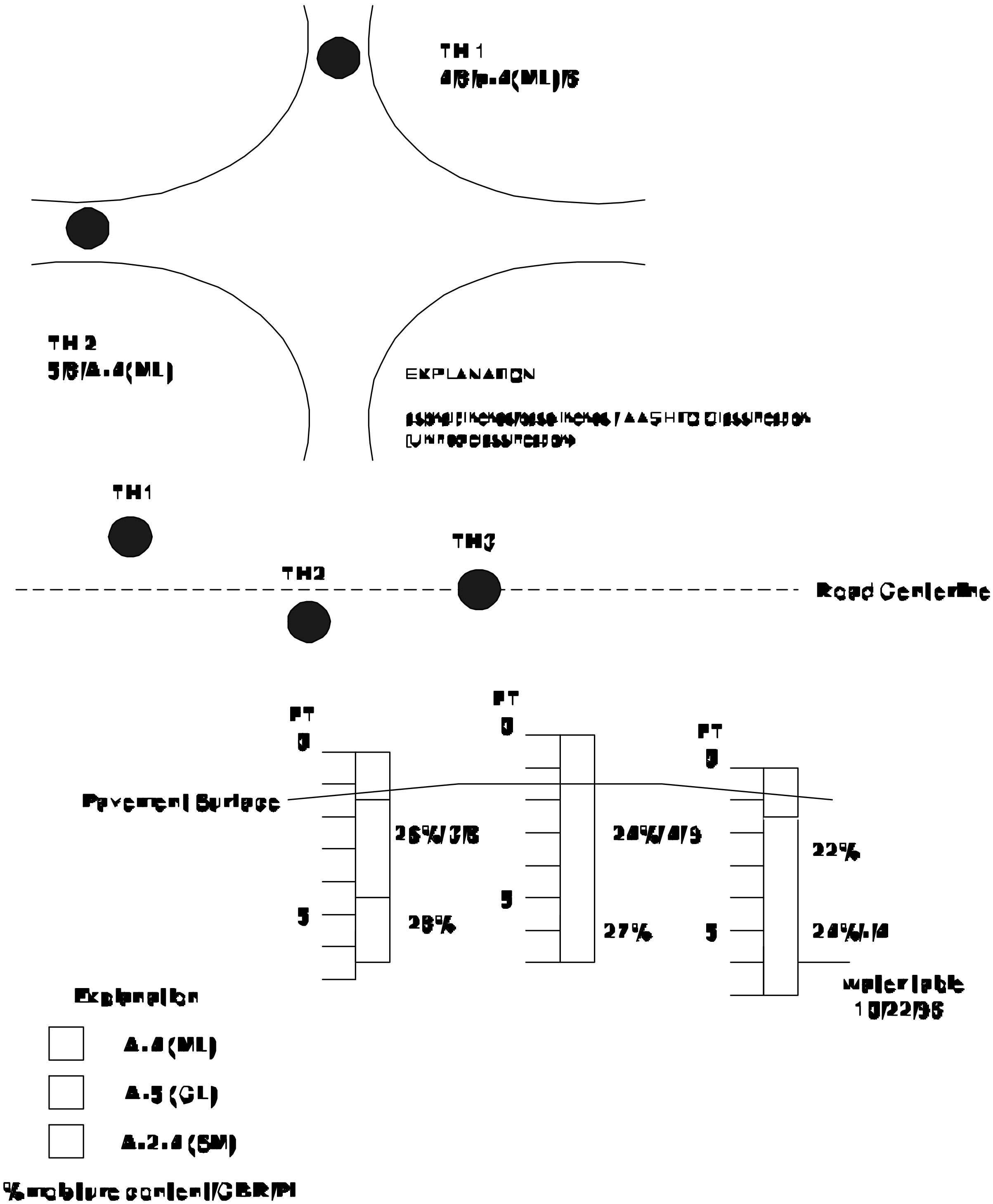
References: USDA Forest Service, Interim Guide for Thickness Design of Flexible Pavement Structures, FHS 7709.11, Chapter 50, Region 6 Supplement No. 20, 1974, Portland Cement Association Publications

Figure 3A-1



Comparison of AASHTO and Unified Soil Classification Systems

Figure 3A-2

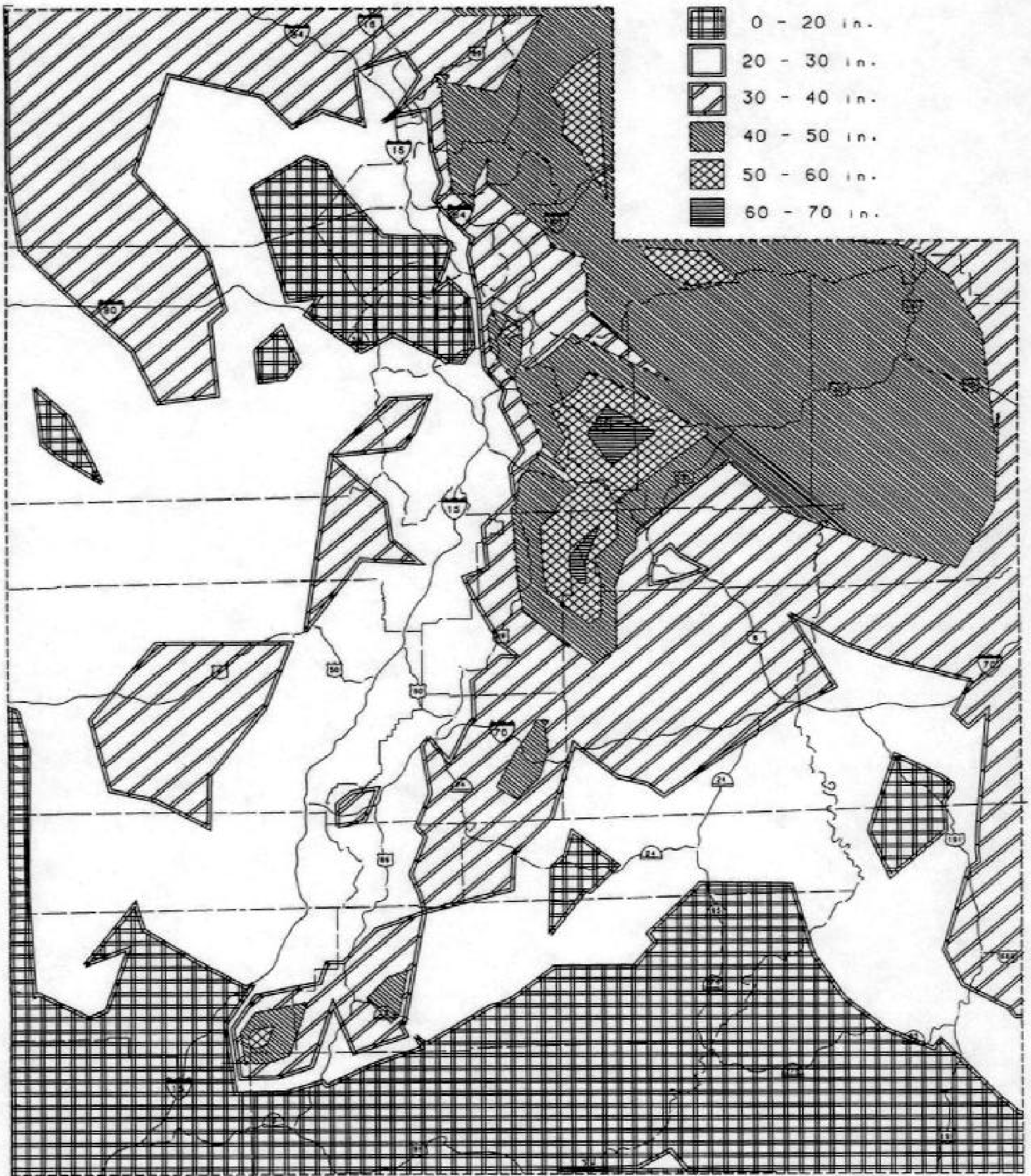


Example Subsurface Summaries

Figure 3A-3

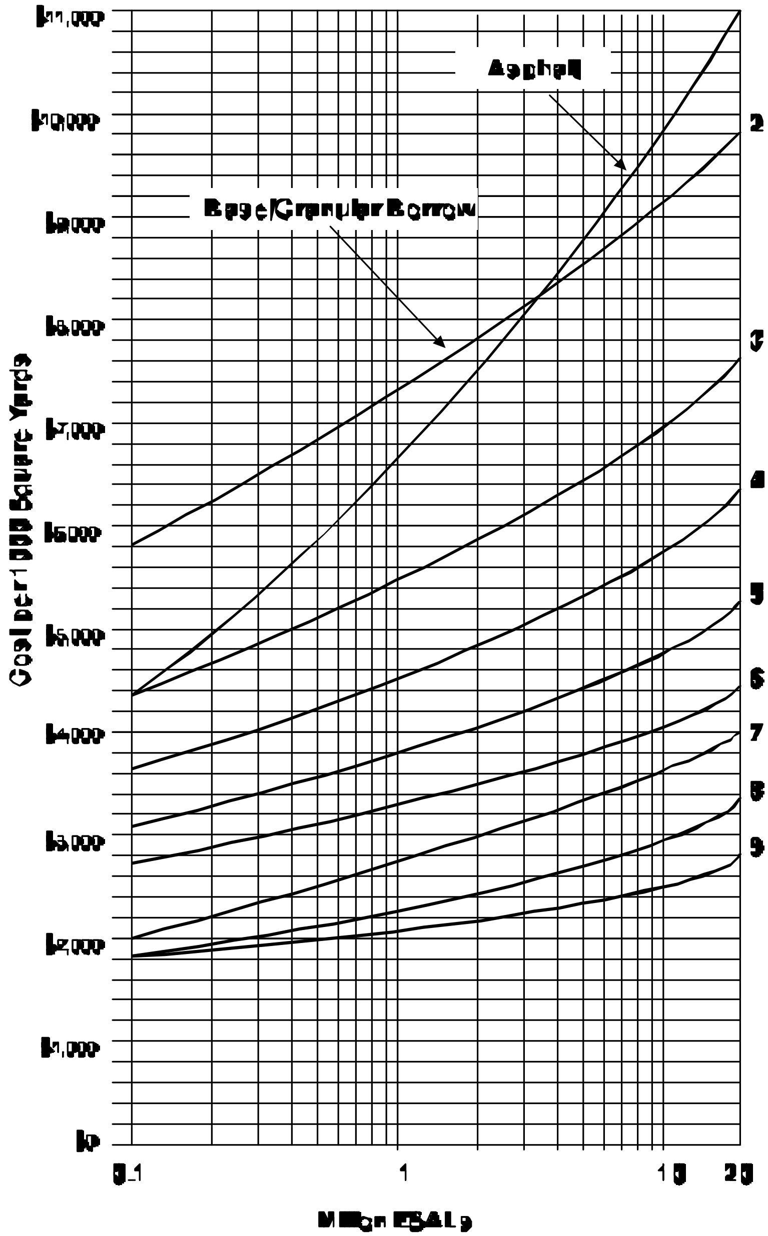
MAX. FROST PENETRATION DEPTH

Estimated Using Freezing Indices (Corps of Engineers Method)



Approximate Pavement Construction Costs for Varying Subgrade CBR's

The primary benefit from a site investigation is assurance of adequate long term pavement performance. However, the ratio of the investigation cost to pavement cost may provide some perspective as to typical investigation costs; Figure 3A-5 shows an approximate relationship of pavement cost to traffic loading and CBR values. For small projects, a higher cost ratio is typical because of the fixed overhead costs including mobilization. As the size of the project increases, the relative cost of site investigations is expected to decrease. For some small projects, it may be more cost effective to use conservative subgrade support values for design than do extensive investigations. Figure 3A-5 includes an example of evaluating the cost effectiveness of an investigation by comparing the construction cost of base/granular borrow thickness determined using a conservative subgrade support value compared to the cost of a thickness based upon a more probable and higher subgrade support value that would require additional investigation costs.



Example Use:

Reading Chart:

For traffic loading of [illegible] million ESALs and a subgrade CBR of [illegible], cost per 1,000 square yards surface area approximately [illegible] for the Asphalt layer and [illegible] for the underlying base and granular borrow. Total construction cost of pavement is approximately [illegible] per 1,000 square yards surface area.

Potential Value of Testing:

ESALs = [illegible] million
Conservative CBR estimate = [illegible], for this CBR cost of base/granular borrow per 1,000 square yards = [illegible]
Anticipated CBR based on experience = [illegible], for that CBR cost of base/granular borrow per 1,000 square yards = [illegible]
Investigation and testing could save [illegible] per 1,000 square yards.

Assumptions for developing graph:

Unit Costs	$/inch thickness
Asphalt	[illegible]
Untreated Base	[illegible]
Granular Borrow	[illegible]

Thickness used in cost estimating are based on [illegible]% design reliability. Minimum granular borrow or base thickness is [illegible]". Thickness/cost of asphalt only varies with ESALs because base support value is constant.

Approximate Pavement Construction Costs for Varying Subgrade CBRs

Figure 3A-5

Section 3B

Traffic Data Analysis for Pavement Design and Evaluation

Introduction:

The Utah Department of Transportation follows FHWA's HPMS (Highway Performance Monitoring System) Requirements. This HPMS system requires the department to set up a statewide sampling frame. The volume is collected at over 5000 sites, with one third collected each year. At three sites the volume is collected for a minimum of 48 hours at each site. From these volume sites, 300 random sites were selected for a traffic classification count. Again counts are performed over a three year period, with 100 of these classifications counts performed per year. Then in order to meet HPMS's requirements for collecting WIM data, 90 sites were selected out of the 300 classifications sites in order to calculate truck factors for functional classes. It is this sampling process which produces the design factors and growth rates for volume, vehicle classifications and weigh-in-motions data for calculating equivalent single axle load factors for each vehicle classification to be used in our design process.

All project traffic information will be requested through the Utah Department of Transportation for all state routes where traffic information is available. These will be written requests using the Traffic Request Form, Table 3B-1. The requests will be through your Regional Pavement Management Engineer via the Planning Division's statistical engineer. The traffic information will be supplied in most cases by the department, but there will be a number of limited projects that UDOT will not have the desired traffic information. It will be these projects that require manual project design ESAL calculations to be performed by the design consultant. If the design ESAL calculations are to be performed by a consulting agency, it will be coordinated and approved through the Regional Pavement Management Engineer. The Regional Pavement Management Engineer will meet with the consulting agency during a mandatory "initial consultant design concept meeting" to discuss many design considerations, one consideration being the traffic calculations. The initial concept meeting will include the Regional Pavement Management Engineer, design consultants, and other UDOT representatives to discuss project specific details. The consultant will submit a letter to the Regional Pavement Management Engineer, outlining the pavement design details agreed upon in the initial consultant design-concept meeting for approval. This approval is project specific and will be done on a project by project basis. A pavement design performed without an approved traffic collection and extrapolation methodology, may be rejected. The approved method may contain one or more of the methodologies defined or discussed in the traffic data analysis section of this manual.

Analysis Period

The analysis period refers to the total period of time which will cover the “design life” of the particular pavement design strategy. When consultants perform pavement designs for UDOT, the “Design life” will be determined at the initial concept meeting by the UDOT Regional Pavement Management Engineer. The “design life” will be provided by the Regional Pavement Management Engineer during the initial concept meeting. This performance period must take into account many design strategy considerations other than just the pavement section in question, which may or may not be known to a design consultant. The Regional Pavement Management Teams determine pavement design life by incorporating the knowledge of planned future corridor projects, future widening and overlay projects, safety projects as well as information from other management systems besides pavement management, while still balancing the design and capacity needs with funding.

Manual Traffic Counts:

There is a chance that on some of UDOT’s Local Government projects, there could be volume information provided by HPMS. However, as previously discussed in the introduction section, there will be pavement design projects that UDOT does not possess the traffic information required in order to calculate the project specific accumulative design period Equivalent Single Axle Loads. Manual counts could be required in these instances because they could be the only manner available to calculate the percent of truck traffic on any given pavement section.

Note: Keep in mind that many local roads won’t have a great deal of truck traffic. On some routes, such as I-15, it could be possible to need as many as three people to collect data on a manual count. This would be a rare case where UDOT wouldn’t have traffic data on a route with three lanes in one direction.

1. **Volume**: The volume should be collected using tubes or permanent counters over a minimum of 48 hours period of time. These short time counts need to be adjusted to an AADT (Average Annual Daily Traffic). UDOT’s Planning Division can provide the factors.

2. **Classification:** A manual classification count would probably be necessary in order to collect classification information. WIM equipment could be used if it is available, the use of two tubes, or the use of Permanent Traffic Recorders (ATR’s classify by length). The classification requires a minimum of a 7 hour count, making sure it includes either an a.m. or p.m. peak hour. Again, these short time counts require adjustments to an annual average. Contact UDOT’s Planning Division for the factors.

3. **Growth Rates (for design year traffic)**: Contact UDOT's Planning Division to obtain the growth factors needed to extrapolate the 48 hour volume data and the 7 hour classification data into a full one year traffic count (current design year traffic count).

4. **Growth Rates (For Volume, Classification & Axle Configuration)**: The Regional Pavement Management Engineer can give some guidance, when needed, in the areas where there are no growth rates available for volume, classification and axle configurations. When the truck factors (ESALs/vehicle) are not available, use the Utah 91-93 Equivalent Single Axle Loads. See Table 3B-1.

Vehicle Volume and Classification:

Network volume and classification data are generally compiled according to type of area (e.g. rural or urban), functional classification (e.g., interstate, principal arterial, minor arterial, or collector), and volume group as defined by AASHTO highway functions. While this type of data may be adequate for planning and preliminary design purposes, it may produce gross ESAL estimation errors when used for the design or evaluation of individual projects. When a consultant is providing project specific traffic data, the type of data collected will be determined in the initial concept meeting.

Project-specific data that is collected over a short period of time must be adjusted for hourly, daily, and monthly or seasonal variations, as determined from continuous count data. These variations include, but are not limited to highway functional class, geometric type, vehicle volume, vehicle classification, vehicle axle configurations and growth rates for all vehicle information types.

Vehicle Classification:

Vehicles are typically classified according to whether or not they carry passengers; further classification is based upon gross weight and/or axle configuration (e.g. three-axle single unit trucks, six axle multi-trailer trucks, etc.). The FHWA has adopted the following 13 vehicle classifications which will be grouped into five categories to be used in the process of calculating design traffic when design traffic is not available through UDOT.

Vehicle Classes:

Category One: (general two axle vehicles)

1. Motorcycles
2. Passenger Cars
3. Other Two-Axle, Four-Tire Single-Unit Vehicles

Category Two (buses)

4. Buses

Category Three: (Single unit trucks)

5. Two-Axle, Six-Tire Single-Unit Trucks
6. Three-Axle Single-Unit Trucks
7. Four or More Axle Single-Unit Trucks

Category Four: (single trailer trucks)

8. Four or less axle single trailer combinations
9. Five-axle single trailer combinations
10. Six or more axle single trailer combinations

Category Five: (multi-trailer trucks)

11. Five or Less Axle Multi-Trailer combinations
12. Six-axle multi-trailer combinations
13. Seven or more axle multi-trailer combinations

The FHWA recommends that all traffic data should be collected according to these 13 categories as the “standard.”

Equivalent Single Axle Loads:

One principle of pavement design is that different wheel loads and load configurations produce different stresses and strains in various layers of a pavement structure, with larger and more concentrated loads producing larger stresses and strains. The repeated application of these stresses and strains causes load-related pavement deterioration (fatigue). Observations of the development of load related distresses have led to the development of equivalent single axle loads factors (ESAL factor). An ESAL Factor is the ratio of the number of applications of a given load and axle configuration required to produce the same deterioration as a number of applications of a standard load and axle configuration. ESAL factors vary according to pavement type (flexible vs. rigid), pavement structure (structure number or slab thickness), axle load, axle configuration (single vs. Tandem vs. Others), tire pressure, and terminal serviceability level, serviceability loss as well as route functional class.

NOTE: ESAL Factors form the basis of traffic analysis for the structural design of pavements. (See table 3B-2, for ESAL calculations) (for functional classification identification, see table 3B-3)

ESAL Computations:

Table 3B-2 can be used to compute historical traffic levels or forecast ESAL's for a given pavement section. Procedures for completing this table and the required computations are described below: The growth factors for vehicle volume, vehicle classification and axle configuration shall be included in the initial concept meeting approved design concept plan.

1. Average daily traffic volume for the base year (first year) in the design/analysis period should be entered in column A for each vehicle classification or group being considered in the traffic analysis, representative of the design location.

2. A growth factor must be computed or selected from Table 3B-4 for each vehicle classification or group and entered in column B. Each growth factor is a function of the design/analysis period and the annual growth rate of the vehicle classification of interest. These growth factors vary from one vehicle type to another and they must be taken into account in the calculations.

3. Column C is obtained by multiplying column A by column B by 365 (days per year). This column contains the estimated number accumulated applications of specific vehicle types during the design/analysis period. (Classification and volume growth rates must be considered.)

4. The truck load factor (ESAL per truck) for each vehicle classification/group should be entered in column D. These numbers are taken from table 3B-5 (Distribution and directional Factors). (ESAL factor growth rate must be taken into consideration.)

5. Column E is the product of column D and Column C and represents the total number of ESAL's applied by each vehicle classification/group during the design/analysis period. Summing the figures in column E produces the total number ESAL applications that have been or will be applied during the analysis/design period. This number must be corrected for directional and lane distributions if such corrections were not incorporated in the volume estimates in column A (See top of table 3B-5).

PROJECT TRAFFIC DATA
REQUEST FORM

State Route Number __________ Project Number (If programmed): __________

Project Description: __________

Milepost-Begin: __________ End: __________

Construction Year: __________

Number of Lanes (after construction -both Directions): __________

Design Period (check all that apply)

5 years ______ 10 years ______ 20 years ______ 30 years ______

40 years ______ ______ years ______

Project Scope (Check any potential strategies):

Overlay ______ Reconstruction ______ Widen - Add Lanes ______

New Alignment ______ Bridge Replacement ______

Inlay ______ Recycle ______ Other ______

Pavement Type:

Bituminous ______ Concrete ______ Both Required ______

Special Requirements or Comments: __________

____

____

Requested By: __________

Date of Request: __________

Table 3B-1

ESAL Calculations

Project Description:

State Route: ___________
Beg. M.P.: ___________ **End M.P.:** _____________
Project Scope: ___________ **Region:** _____________
Rigid or Flexible: ___________
Construction: ___________ **Functional Class:** _____________
Design Period: ___________ **SN:** _____________

Vehicle Types	Current Traffic (A)	Growth Factors (Table 3B-4) (B)	Design Traffic (Columns A*B) (C)	ESAL Factor (Table 3B-5) (D)	Design ESAL (Columns C*D) (E)
Category I Axle Class 1-3					
Category II Axle Class 4					
Category III Axle Class 5-7					
Category IV Axle Class 8-10					
Category V Axle Class 11-13					
Total Design ESALs					

Table 3B-2

UDOT's Functional Classification Code

Rural	
01	**Interstate System**
02	**Other Principal Arterials**
06	**Minor Arterial System**
07	**Major Collector**
08	**Minor Collector**
09	**Local System**
Urban	
11	**Interstate System**
12	**Other Freeways & Expressways**
14	**Other Principal Arterials**
16	**Minor Arterial Systems**
17	**Collector System**
19	**Local System**

Table 3B-3

Annual Growth Rate

Anal. Period Years	No Growth	2	4	5	6	7	8	10
1	1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20	24.3	29.78	33.06	36.79	41.00	45.76	57.28
25	25	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35	49.99	73.65	90.32	111.43	138.24	172.32	271.02
40	40	60.40	95.03	120.80	154.76	199.64	259.06	442.59

Table 3B-4

$$Factor = \frac{((1+g)^{n})-1}{g} \quad where \quad g = \frac{rate}{100} \quad g \neq 0$$

If Annual Growth is zero, the growth factors are equal to the analysis period.

Note: The above growth factors multiplied by the first year traffic estimate will give the total volume of traffic expected during the analysis period.

Lane Distribution and Directional Factors:

Design Lane ESAL's = base year vol. X directional factor x lane factor (F) x 365 days x midpoint truck factor x growth factor.

Where F = 1, for 3 lanes or less

$F = 2.275\ (AADT)^{-0.1054}$ for 4 or 5 lanes

$F = 2.484\ (AADT)^{-0.1312}$ for 6 lanes or more

I = annual growth rate = 10 exp. {[log(f/b]/n}-1

n = design period (yrs.)

growth factor = $\{(1+I)^{n-1}\}/I$

Direction factor defaults to 0.5 for two way and 1.0 for one way traffic if unknown.

Midpoint truck factor = truck factor :Class 1-4

= truck factor + 0.01(n/2) :Class 5-7

= Truck factor + 0.03(n/2) :Class 8-13

Axle Class	Functional Class - *Refer to Table 3B-3					
	1		2		6	
	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	1.22	0.88	1.28	0.88	0.83	0.88
5-7	0.5064	0.4718	0.2065	0.1996	0.3061	0.2896
8-10	4.8749	2.8744	2.9648	1.7796	2.7212	1.641
11-13	5.2436	3.6942	2.2342	1.3596	2.6832	1.7199

Table 3B-5

Distribution and Directional Factors						
Axle Class	Functional Class - *Refer to Table 3B-3					
	7		8		9	
	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	0.83	0.88	0.83	0.88	0.83	0.88
5-7	0.3103	0.2798	0.3103	0.2798	0.3103	0.2798
8-10	1.5930	1.0079	1.5930	1.0079	1.5930	1.0079
11-13	0.8221	0.6400	0.8221	0.6400	0.8221	0.6400

Axle Class	Functional Class - *Refer to Table 3B-3					
	11		12		14	
	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	1.2	0.88	1.2	0.88	1.08	0.88
5-7	0.3827	0.3529	0.3827	0.3529	0.1993	0.1912
8-10	2.8934	1.6884	2.8934	1.6884	3.0104	1.8133
11-13	3.6508	2.5203	3.6508	2.5203	3.1270	1.9288

Table 3B-5

Distribution and Directional Factors

Axle Class	Functional Class - *Refer to Table 3B-3					
	16		17		19	
	Rigid	Flexible	Rigid	Flexible	Rigid	Flexible
1-2	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
3	0.03	0.03	0.03	0.03	0.03	0.03
4	1.07	0.88	1.07	0.88	1.07	0.88
5-7	0.3827	0.3529	0.3827	0.3529	0.1993	0.1912
8-10	4.2476	2.6028	4.2476	2.6028	4.2476	2.6028
11-13	5.2762	3.3584	5.2762	3.3584	5.2762	3.3584

Table 3B-5

Section 3C
Rigid Pavement Design Procedures

Introduction

This section of the design manual deals with UDOT's rigid pavement design procedures. These procedures are based on the AASHTO rigid design equation, using specific parameter values based on climate considerations and Department objectives. The procedures contained within this section are intended to assist in completing a rigid pavement design in accordance with UDOT practice. The design process starts with an initial design-concept meeting between the Regional Pavement Management Engineer, and the pavement designer. The concept of the pavement design is discussed and the designer then collects the necessary engineering data, performs the design and then submits the design for approval.

UDOT Objectives

Within UDOT's rigid pavement design process, there are 5 major objectives that are used to complete the design process. These objectives are tailored to UDOT's concepts and requirements for pavement design.

1. **Determine the pavement design concept.** The concept is based on location (rural/urban) and pavement function (collector, arterial, interstate, etc.). The objective is accomplished during the initial design-concept meeting with the Regional Pavement Management Engineer. The pavement design concept will affect issues such as load transfer requirements, design life, number of design sections, and non-erodible base requirements.

2. **Design rigid pavement section based on traffic (structural) requirements.** The objective is attained through collection of project and site specific data and UDOT supplied information, which is then used in the AASHTO design equation. Project and site specific data includes such things as soils information, falling weight deflectometer data, traffic data, pavement distress data, corridor concepts, budgetary constraints, etc.

3. **Make adjustments for minimum layer thicknesses and minimum frost depths, and perform pavement drainage design.** Using UDOT's required minimum thicknesses and standard frost depth charts, the structural section design is revised to accommodate

construction and climate conditions. Drainage designs for removing water from the pavement layers are then addressed.

4. **Evaluate design through cost analysis and design merits.** The objective entails reviewing the selected design section and discussing the reasoning for selecting the design based on life cycle cost and engineering merit.

5. **Address material specifications and construction requirements specific to the project.** Additional requirements that are related to project special provisions or construction processes are addressed and highlighted to assist field construction crews.

PCCP Types

There are three types of rigid Portland cement concrete pavements that are considered typical industry standards: plain jointed concrete pavement (PJCP), jointed reinforced concrete pavement (JRCP) and continuously reinforced concrete pavement (CRCP). Utah prefers plain jointed concrete pavement for its roadways, with load transfer as required. The concepts and procedures within this manual are based on the current AASHTO procedure related to designing PJCP. The design requirements for JRCP and CRCP will not be presented and the design of JRCP or CRCP will not be accepted without prior approval from the Regional Pavement Management Engineers during the initial design-concept meeting.

Base Materials

The typical section configuration for rigid pavements in Utah includes the use of three different base layers. The first subbase layer consists of *Granular Borrow* material, which is an A-1-a, non-plastic material and is placed on the subgrade. A geotechnical separator fabric can be used in between if needed. Granular borrow gradation requirements include a 3 inch maximum particle size. The second subbase layer consists of an *Untreated Base Course* material which is also an A-1-a, non-plastic material, but the maximum particle size can be chosen as 1 inch or 3/4 inch, with dependant gradations. The base layer consists of a *Lean Concrete Base* material which is a low-strength, ready mix material that is used as a non-erodible base and paving platform.

Variations from the typical configuration can include the use of an *Asphalt Treated Base* material under the PCCP, the placement of the PCCP directly onto the untreated base course material or the use of gap gradations for any or all of the base layers to provide proper drainage characteristics. Variation from the use of the standard base layers will be addressed on a project by project basis and will be determined by the Designer and Regional Pavement Management Engineer during the initial design-concept meeting.

Structural Thickness

The structural thickness requirement for PCC pavement in Utah is based on the AASHTO rigid design equation which incorporates traffic, climate, soils, materials and statistical characteristics. Modifications are made to the input parameters to tailor the design to Utah's needs. The typical rigid pavement section consists of a Portland cement concrete pavement layer (minimum 9 inches), 4 inches of lean concrete base, 6 inches of untreated base course, and a minimum of 12 inches of granular borrow. The AASHTO concepts and the modifications are discussed in the following sections.

Parameters

The parameters discussed are based on the AASHTO design requirements and input parameters. The discussions include typical Utah values for each parameter and the engineering principles used in their determination. Variation from the values is permitted with proper engineering principle as the basis and with concurrence by the Regional Pavement Management Engineer.

Pavement Classification

Knowledge of pavement classification and function is necessary to determine the number and type of base layers, drainage requirements and load transfer requirements for the pavement. General guidelines on the variation from the standard sections are outlined below:

Urban

- may not need lean concrete base course
- may not require load transfer devices

Rural

- pavement drainage may be accomplished through minimum permeabilities of day lighted layers or through periodic cross-drains in lieu of lateral edge drains and full collector systems

Interstate

- high volume, high load, full design typically required as a minimum with additional considerations

Design Life/ ESAL Requirements

The Regional Pavement Management Teams determine pavement design life by incorporating knowledge of planned future corridor projects, future widening and overlay projects, safety projects as well as information from the other management systems. Balancing of the structural design and capacity needs with funding limitations plays a large part. The design life used for the pavement design must be verified before beginning the design in the initial design-concept meeting with the UDOT Regional Pavement Management Engineer.

The standard design life for a PCC pavement in Utah is a minimum of 40 years. ESAL requirements for the structural design are based on the total number of rigid ESALs that the pavement design lane will carry throughout its design life. The design lane is the single direction lane that carries the majority of the traffic load. The design lane is typically the outside lane for multi-lane highways. Traffic and ESAL information is obtained from UDOT Planning, as discussed in the traffic section of this manual. ESAL loads for a PCC pavement should be based on 40 years of projected rigid ESALs. Variations must be discussed with and approved by the Regional Pavement Management Engineer.

Serviceability

As defined in the flexible pavement design section, the serviceability of a pavement is its ability to serve the type of traffic that uses the facility. The pavement's serviceability is measured with the Present Serviceability Index (PSI), which ranges from 0 (impassible) to 5 (perfect).

UDOT has set the initial and terminal Serviceability requirements for rigid pavements at 4.5 for $PSI_{initial}$ or post construction and 2.5 for $PSI_{terminal}$ for all pavements. ÄPSI is defined as:

$$\text{ÄPSI} = PSI_{initial} - PSI_{terminal} = 4.5 - 2.5 = 2.0$$

Hand calculation of the AASHTO equations will require the use of ÄPSI and $PSI_{terminal}$, while most computer software solutions will require the initial and terminal points.

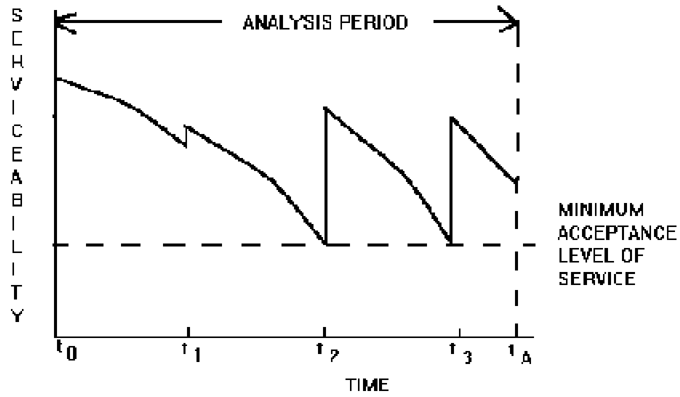


Figure C-2.1. Typical Serviceability Curves.

Rupture Modulus

UDOT has set its rupture modulus value (S'_c) based on minimum flexural strength requirements of paving quality PCC. For all designs, S'_c is accepted at 650 psi.

Elastic Modulus

The elastic modulus value (E_c) is based on material stress strain characteristics and is a function of the compressive strength of the Portland cement concrete. The relationship between the elastic modulus value and the compressive strength is estimated at

$$E_c = 57{,}000\,(f'_c)^{0.5}$$

Typical values range from 3,500,000 psi to 4,500,000 psi, dependant on the strength characteristics. Utah has set the E_c requirement for the AASHTO equation at 4,000,000 psi.

Design k-value

The design k-value and the projected traffic loadings are the two major project specific influences in the UDOT design procedure. The traffic projections are based on future plans, while the design k-value is based on the existing conditions of the subgrade along with the probable makeup of the future pavement section. The design k-value is a measure of the ability of the base structure of the pavement to support the PCC pavement slab and is related to the Moduli of the subgrade and base layer materials.

Starting with a CBR value for the existing subgrade soils, an initial design k-value can be determined using figure 3C-1. This figure can then be improved based on the type and thickness of the base layers of the pavements. Using figures 3C-2 through 3C-5, enter the appropriate graph at the k-value of the layer below, and using the proper depth curve for the base layer, determine the adjusted design k-value above the new base layer. This process can be repeated for each successive base layer placed between the subgrade and the PCC pavement. The final step is to adjust the k-value for loss of support based on the characteristics of the layer immediately below the PCC pavement. Using table 3C-1 and figure 3C-6, determine the corrected k-value immediately below the PCC pavement layer. A typical loss of support correction value is obtained from table 3C-1 and the value is used to determine which line use for k-value correction in figure 3C-6. The final k-value at this point is the value used for design of the slab thickness. The minimum value required for the k-value is 450 pci. For urban roadways with light traffic, where cure time for lean concrete base is problematic, the minimum design value for the k-value may be reduced to 350, and the pavement can be designed without a treated base course. This approach is applicable only to projects with less than 1 million design life ESAL's, and the process should be discussed with the Region Pavement Management Engineer. If the design value is less than 450 pci, then additional base material must be included, sufficient to raise the k-value above 450. The maximum values allowed for the k-value are 800 pci for untreated materials (UTBC, GB, FDGB) and 1200 pci for treated materials (LCB, ACB, PLCB).

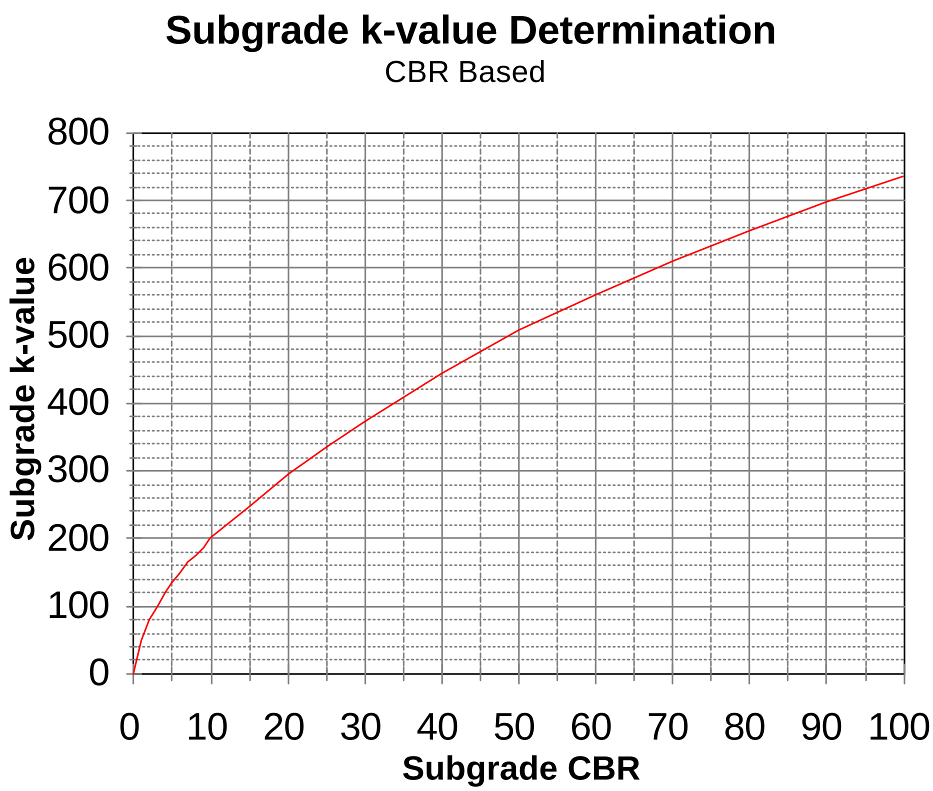


Figure 3C-1

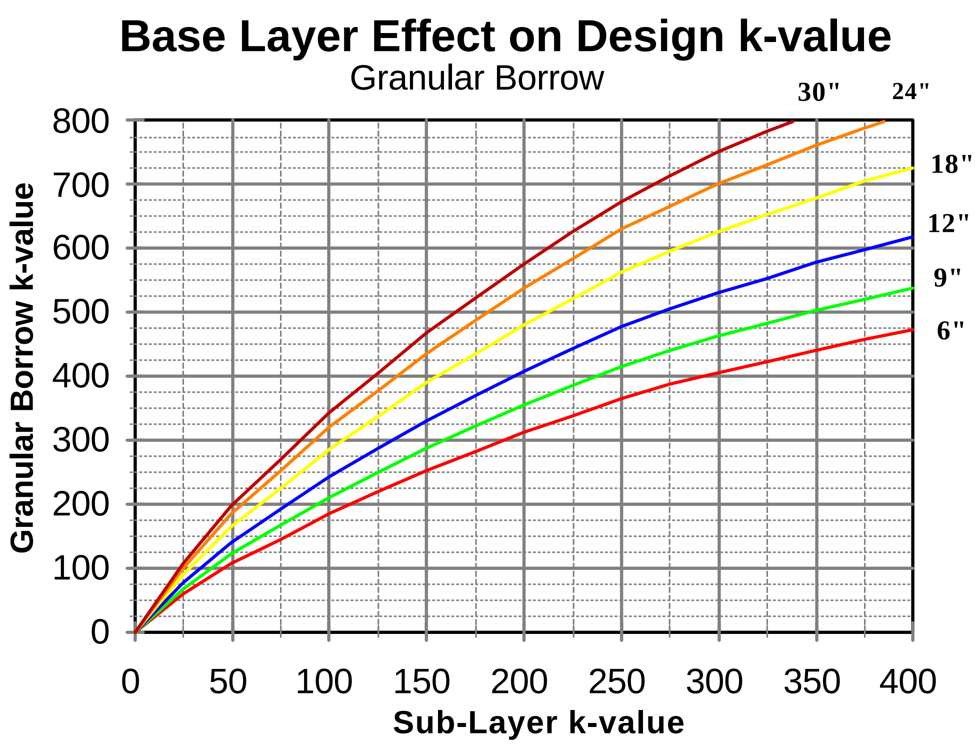


Figure 3C-2

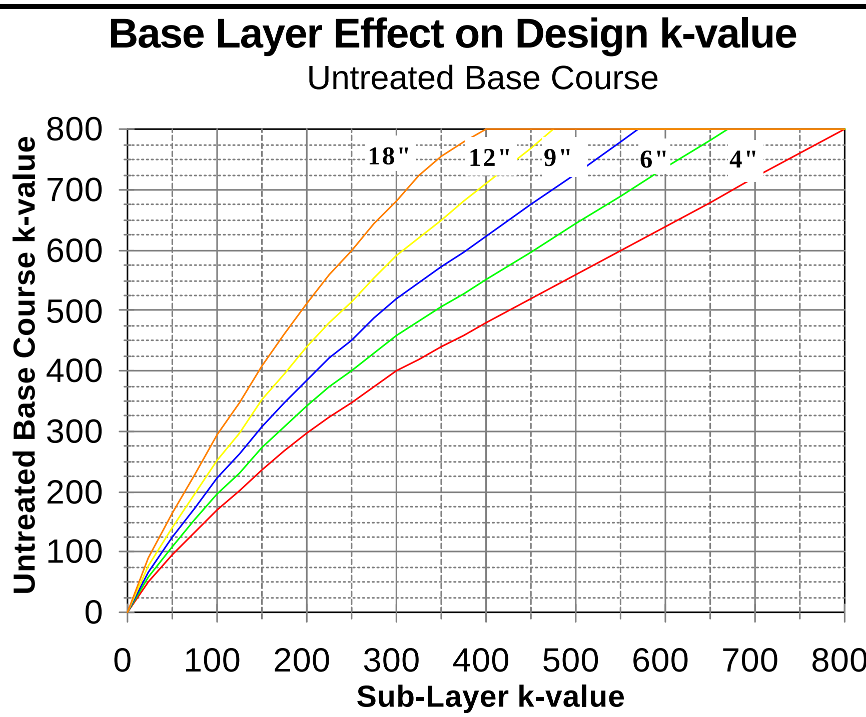


Figure 3C-3

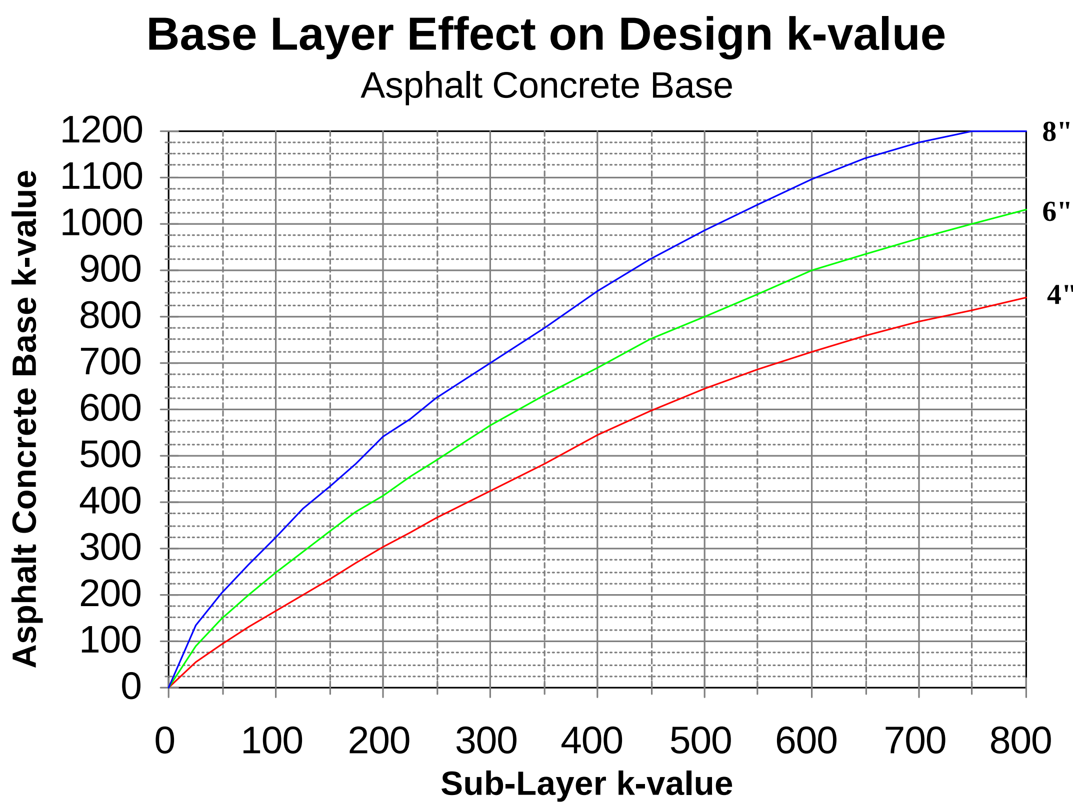


Figure 3C-4

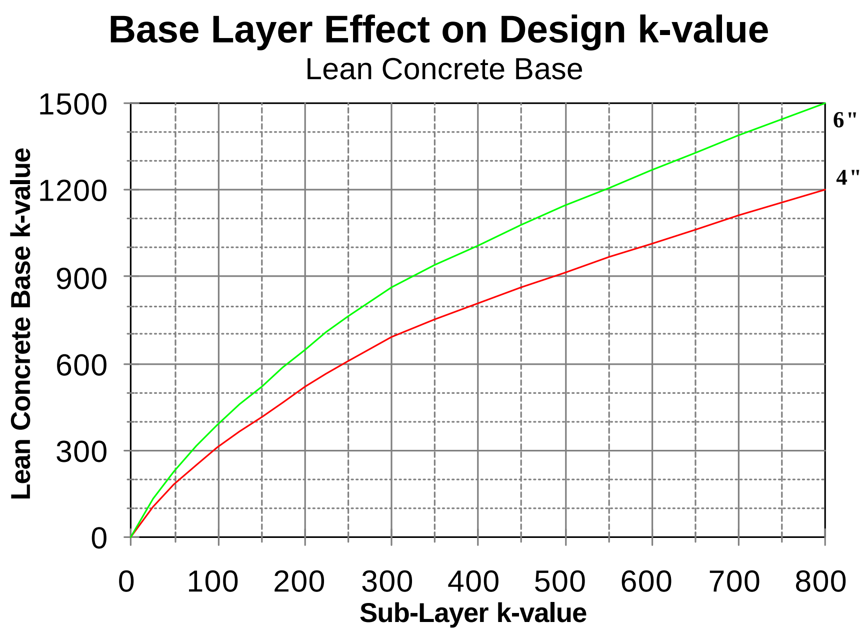


Figure 3C-5

Loss of Support Values for Typical Pavement Base Layer Materials

Type of Material	Loss of Support Value (LS)
Lean Portland Cement Bases *(LCB, PLCB)*	0.0
Asphalt Cement Bases *(ACB, PACB)*	0.25
Cement or Lime Stabilized Bases	0.75
Unbound, Well-Graded Granular Materials	1.0
Unbound, Poorly Graded Granular Materials	1.5
Natural Subgrade Soils	1.5

Table 3C-1

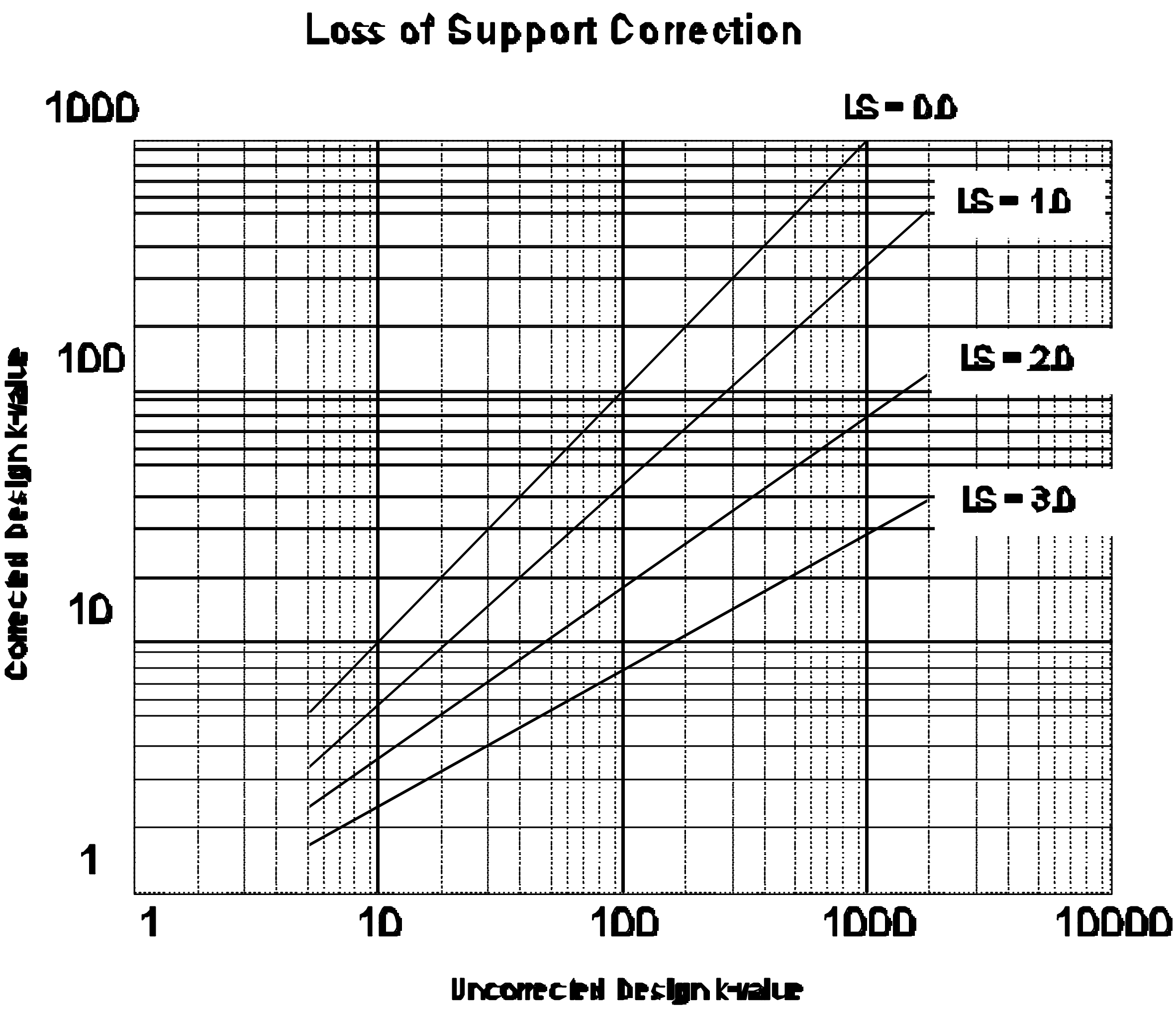


Figure 3C-6

Reliability

The reliability factor in the AASHTO equations indicates the level of certainty in the design, with regard to its ability to withstand the projected structural loadings throughout its design life. As with the flexible design requirements, UDOT has set the reliability factors at 95% for interstate highways and 90% for all others. The values typically entered into equations are in the form of a percent instead of a decimal.

Standard Deviation

The standard deviation parameter controls the level of precision for the designs. For Portland Cement Concrete Pavements, UDOT requires a value of 0.35.

Load Transfer

Load transfer requirements for PJCP can be accomplished with a variety of devices. For typical UDOT projects, these devices are smooth dowels in the transverse joints. Load transfer requirements are a function of the projected traffic load. UDOT has not defined a breakpoint where load transfer is required or which type of device is required. These issues should be discussed with the Regional Pavement Management Engineer during the initial design-concept meeting. Input values for the AASHTO equation are different for pavements that have PCC or ACP shoulders, along with the presence of load transfer devices. Utah requires the use of tied PCC shoulders. Use table 3C-2 as a guide for the selection of this value.

Minimum Values for Load Transfer Coefficient in PJCP

	Load Transfer Coefficient
With Load Transfer Devices	3.0
Without Load Transfer Devices	4.0

Table 3C-2

Drainage Coefficient

The AASHTO rigid pavement design procedure provides a coefficient to take into account the effects of certain levels of drainage on pavement performance. The design engineer must identify the level or quality of drainage that is achieved under a specific set of drainage conditions. The following are the AASHTO definitions for drainage quality.

Drainage Quality Classification

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	water will not drain

Table 3C-3

Drainage Coefficient Values

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	< 1%	1% - 5%	5% - 25%	> 25%
Excellent	1.4 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.05 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Table 3C-4

The maximum value for the drainage coefficient is 1.0 for UDOT pavement design. In poor draining areas or areas that have high water tables or other sources of water, the drainage coefficient may be reduced in accordance with table 3C-4, and should be discussed with the Regional Pavement Management Engineer at the initial design-concept meeting.

Stage Construction

Stage construction implies that the pavement section will be built in several stages over time. UDOT's current policy is to not bank money for future construction. This precludes the use of stage construction opportunities for UDOT projects. Single stage construction should be used for all design projects.

Thickness Calculation

The slab thickness design is performed using the AASHTO rigid design equation with the parameter values as specified within this manual. The four major areas that are project specific are the traffic load requirements, subgrade soil conditions, load transfer coefficient and the drainage coefficient. Each of these areas must be investigated prior to use of the equation. After all of the design variables have been identified, the equation can be solved for the slab thickness. With the complexity of the equation, the use of a computer software program to solve the equation is typical. Hand solutions and software solutions are acceptable. Use of the AASHTO nomograph is acceptable for rough estimates; however, due to the high possibility for 'pencil width' error, the nomograph result will not be accepted as a basis for design. The AASHTO rigid pavement design equation is as follows:

$$\log_{10}W_{18} = Z_R \times S_0 + 7.35 \times \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624\times 10^7}{(D+1)^{8.46}}} + (4.22-0.32PSI_t)\times\log_{10}\left[\frac{s_c' \times C_d[D^{0.75}-1.132]}{215.63\times J[D^{0.75}-\frac{18.42}{(E_c/k)^{0.25}}]}\right]$$

Where:

W_{18} = Number of Design Lane ESALs

S_O = Overall Standard Deviation

ΔPSI = Design Serviceability Loss

S'c = PCC Modulus of Rupture

J = Load Transfer Coefficient

k = Design k-value

Z_R = Z-Table Factor at Reliability R(%)

D = Design Slab Thickness

PSI_t = Terminal Serviceability

Cd = Drainage Coefficient

E_c = PCC Elastic Modulus

The design thickness calculated from this equation is the minimum structural thickness required for the pavement. All thicknesses of PCC pavement should be rounded up to the nearest ½ inch, with consideration given to the need for grinding during future rehabilitation projects. The base layers should be rounded to the nearest inch.

Sections Adjustments for Frost, Layer Thickness and Drainage

After identifying the minimum structural requirements for the pavement, the thicknesses and requirements for each individual layer, along with the overall thickness of the section must be adjusted for environmental and construction conditions. The three conditions that must be addressed are frost, drainage and minimum layer requirements.

Frost

Minimum frost depth requirements apply to the overall thickness of the section. Using the frost depth chart in the site evaluation section (figure 3A-4), determine the minimum frost depth requirement for the project location and make the necessary adjustments to the pavement layers. All designs will require a minimum of 30 inches of depth for frost and environmental considerations. Frost susceptible soils in the subgrade will require additional thickness considerations. Increasing the granular borrow thickness to meet frost and environmental concerns is typical.

Drainage Requirements

Drainage of the pavement structure is critical in Utah due the large number of freeze/thaw cycle experienced during the winter. Pavement structure drainage is accomplished through a system comprised of a combination of edge drains, cross-drains, permeable bases, permeable subbases, catch basins and storm drains and/or geotextile fabrics. The use of a pavement drainage system should be investigated for all pavements and should be discussed during the initial design-concept meeting. Omission of pavements drains may affect the selection of the value for the drainage coefficient, and the option of thicker sections compared to the use of pavement drains should be compared on a life cycle cost level.

The use of permeability requirements for the base layers should also be investigated for all pavements, regardless of the type pf drainage system or the type of subgrade soils. Drainage of surface runoff away from the PCC layer is paramount to the long-term survival of the pavement, along with the need for maintaining base support for the pavement as long as possible. The use of gap graded materials and permeability requirements will remove water with a minimal amount of base material loss. A complete description of the pavement drainage system design is to be included with the pavement design and should include layer permeabilities, size and location of pavement drains, and the estimated sources of water for the pavement.

Minimum Layer Thicknesses

In association with the pavement design concept and objectives of the Department, UDOT has set minimum layer thicknesses for the pavement layers. The thicknesses are based on construction requirements, structural requirements, AASHTO recommendations and material characteristics. After determination of the structural section, the thickness of the individual layers should be addressed, and increased if necessary, according to table 3C-5. These adjustments are in addition to the required structural design thicknesses and reducing one layer to compensate for increasing a second should not be considered without reevaluating the structural design requirements.

Minimum Layer Thicknesses

Layer	*Minimum Thickness (in)*
PCCP	9
LCB, ACB, UTBC	4
PLCB, PACB, GB, FDGB	6

Table 3C-5

Note: No layer should be thicker than the layer beneath it, except the PCCP layer, and if structural requirement necessitates more than 30 inches of GB, then additional depth should be made up in embankment (A-1 through A-4 class material).

Life Cycle Cost Assessment

All design alternatives are reviewed for engineering practicality and for economic considerations. The alternatives that have been carried through structural design and minimum thickness evaluation, should be evaluated for life cycle cost based on annual cost to the Department. The life cycle cost analysis procedures are spelled out in a separate part of this manual.

Additional Design Considerations

Additional design considerations address the construction and materials aspects of the pavement design and include issues such as base material specifications, recycling options, ACP and PCCP material requirements, and possible construction issues. These items should be identified and addressed in the pavement design so that the necessary arrangements can be made to incorporate them into the project. References to the current standard drawings and specifications should be made where applicable.

Base Material Characteristics

All base materials are covered under the current version of the UDOT standard specifications book in their generic form. This includes such things as gradation requirements, quality test requirements, construction limitations, etc. These specifications should be reviewed by the designer for all base and paving materials included in the design, and any deviations from the standard required by the design should be highlighted and addressed. Issues to include are things such as permeabilities, CBRs, gap grading, etc.

Creation and use of project special provisions that modify existing material specifications should follow UDOT guidelines for specification writing and numbering. The Regional Pavement Management Engineer can help with the gathering of necessary information.

Recycling Options

Recycling of the existing pavements into the new pavements can be both economically and environmentally beneficial. The materials division of UDOT has defined some limits for use as guidelines in the design and use of recycled material, as shown in table 3C-6. Use of these limits are acceptable without the need for approval. Proper engineering design, constructions issues and necessary specification modifications should be addressed and detailed within the pavement design.

Percentages of Allowable Recycled Material for Pavement Layers

Layer	*Recycled ACP (%)*	*Recycled PCCP (%)*	*Recycled Agg. Base (%)*
PCCP	0	0	0
ACP	25	25	25
LCB, PLCB	0	40	50
ACB, PACB	25	40	50
UTBC	20	100	100
GB, Embankment	30	100	100

Table 3C-6

Note: All materials incorporating recycled material must still meet all specifications for the virgin material, including CBR and drainage requirements. Options using higher percentages of recycling will be reviewed on a project by project basis. Use of higher percentages of recycled material will require organization and demonstration of acceptable testing procedures and an acceptable QC program by the contractor.

PCC Mix Design Recommendations

PCC mix design issues should be addressed when the pavement design includes such things as high/early strength requirements for construction purposes, abnormal or large aggregate gradations for aiding in load transfer, or any other necessary deviations from the current specifications.

Construction Requirement Recommendations

Construction issues that should be addressed are variations from the standard in areas such as, sealant type, joint configuration, surface texture requirements, vibration and compaction variations, etc. and should be included with the design.

Acceptable References

The following list contains acceptable references and software that can be used for designing pavements. Additional references and software can be submitted to the Regional Pavement Management Engineer for review.

AASHTO 1993 Pavement Design Guide
PCA 1990 Design and Control of Concrete Mixtures
TRB 1994 Highway Capacity Manual

DARWin Pavement Design Software
ACPA Pavement Analysis Software

Appendix 3C

Abbreviations

For clarification, the following abbreviations are included to minimize confusion related to the concepts and procedures addressed within this section of the design manual.

PCCP	-Portland Cement Concrete Pavement
PJCP	-Plain Jointed Concrete Pavement
LCB	-Lean Concrete Base
PLCB	-Permeable Lean Concrete Base
ACB	-Asphalt Concrete Base
PACB	-Permeable Asphalt Concrete Base
UTBC	-Untreated Base Course
GB	-Granular Borrow
FDGB	-Free Draining Granular Borrow
PSI	-Present Serviceability Index (initial and terminal)
M_R	-Subgrade Soil Resilient Modulus
SN_c	-Modulus of Rupture for PCC (psi at 28 days)
E_c	-Elastic Modulus of PCC Slab (psi at 28 days)
k	-k-value, design value indicating support capabilities of pavement base structure

Section 3D
Flexible Pavement Thickness Design

Introduction

One of the major objectives of the AASHO Road Test was to provide information that could be used to develop pavement design criteria and procedures. This objective was met with the development and circulation of the AASHO Interim Guide for the Design of Rigid and Flexible Pavements[1] in 1961, which contained design procedures based on empirical models derived from data collected at the AASHO Road Test. After the "Guide" had been used for several years, the AASHTO Design Committee evaluated and revised the Interim Guide in 1972[2] and again (for rigid pavement applications) in 1981[3].

Further evaluations of the "Guide" were undertaken in 1983, and it was determined that although the Guide was still serving its main objectives, some improvements could be made to incorporate advances in pavement design and analysis technology that had been made since 1972. Thus, in 1984-85 the Subcommittee on Pavement Design and a team of consultants revised the existing guide under NCHRP Project 20-7/24 and issued the version entitled AASHTO Guide for the Design of Pavement Structures -- 1986[4].

The 1993 edition of the "Guide[5]" contains only one Volume. This Volume replaces the 1986 "Guide" Volume 1 and serves the same purpose. The major changes included in the 1993 "Guide" are changes to the overlay design procedure and the accompanying appendices L, M, and N. Volume 2 of the 1986 "Guide" is still applicable to most sections of Volume 1 of the 1993 "Guide".

The current "Guide" retains the modified AASHO Road Test performance prediction equations as the basic models for use in pavement design. Major flexible pavement design procedure changes have been made in several areas, including the following:

1. Incorporation of a design reliability factor (based on a shift in the design traffic) to allow the designer to use the concept of risk analysis for various classes of highways.

2. Replacement of the soil support number with the resilient modulus (AASHTO test method T274) to provide a rational testing procedure for defining materials properties.

3. Use of the resilient modulus test for assigning layer coefficients to both stabilized and unstabilized material.

4. Provision of guidance for the construction of subsurface drainage systems and modifications to the design equations to take advantage of improvements in performance that result from good drainage.

5. Replacement of the subjective regional factor with a rational approach to the adjustment of designs to account for environmental considerations such as moisture and temperature climate considerations, including thaw-weakening and other seasonal variations in material properties.

The new “Guide” also includes recommendations and guidelines for conducting economic analysis of alternative designs and a summary of the latest concepts concerning the development and use of mechanistic-empirical design procedures.

Design Assumptions and Procedures

The general applicability and accuracy of any empirical pavement design approach is governed by the selection of the independent variables and their ranges in the experimental design, field conditions and random variability, simplifying assumptions used in the analysis procedures, and the analysis techniques themselves. The AASHTO procedure for flexible pavement thickness design is no exception.

Specific Conditions of the AASHO Road Test

The location of the AASHO Road Test was near Ottawa, Illinois, about 80 miles southwest of Chicago. The facility was constructed along the alignment of Interstate Route 80. This site was chosen because:

1. The soil within the area was uniform and of a type representative of that found in large areas of the country.

2. The climate was typical of that found throughout much of the northern United States.

3. The pavement construction work could ultimately be incorporated in the construction of sections of Interstate 80.

The climate of the Road Test area is temperate with an average annual precipitation of about 34 inches. The average depth of frost penetration is about 28 inches. The AASHTO designation A-6/A-7-6 soils (CBR = 2 - 4) found at the site are generally poorly drained and typically retain more precipitation than can evaporate, thus yielding a positive Thornthwaite
Moisture Index of about 30. The modulus of subgrade reaction (k) after the spring thaw is typically about 45 psi/in.

The test facilities consisted of six two-lane test loops, located as shown in the AASHO Road Test Layout Figure below. The north tangent of each loop was constructed of flexible pavement sections and the south tangent was constructed of rigid pavement sections. Most of the 234 flexible pavement structural design sections (468 test sections, 160 feet in length) comprised a complete replicated factorial experiment investigating the effects of varying thicknesses of surfacing (1, 2, 3, 4, 5, and 6 inches), base course (0, 3, 6, and 9 inches), and subbase (0, 4, 8, 12, and 16 inches). Several additional studies were also conducted to evaluate surface treatments, shoulders and four different types of base course (crushed stone, gravel, cement-treated gravel and bituminous-treated gravel). See the following layout:

AASHO Road Test Layout

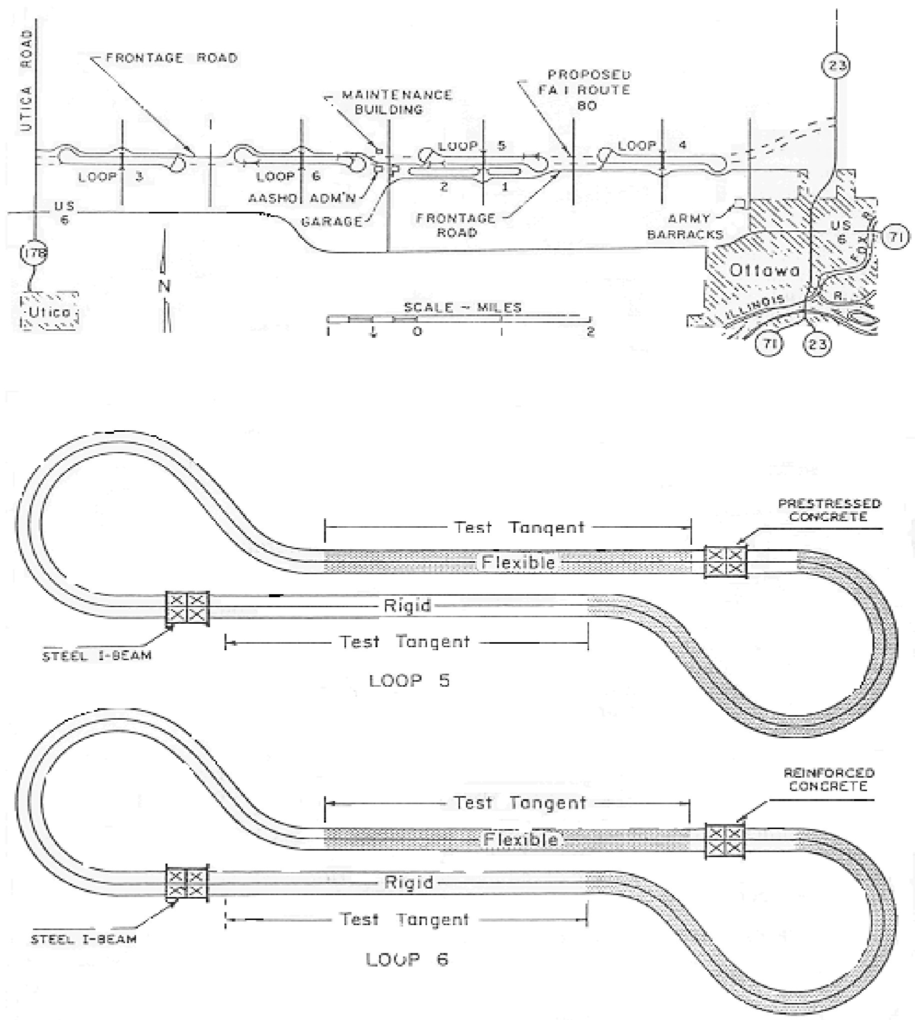


Figure 3D-1

Although conventional construction techniques were used, construction was of extremely high quality because of close supervision and on-site materials testing by the Illinois Division of Highways and the Highway Research Board staff. In addition, an extraordinary effort was put forth to insure uniformity of all pavement components. For example, no construction equipment other than that necessary for compaction was permitted to operate in the center 24-foot width of the roadway, and all turning operations on the grade were limited to specially designated transition areas. Therefore, variations in concrete, aggregates, moisture, density, subgrade soil properties, etc., were much lower than can be expected in most normal highway construction.

No traffic operated over Loop 1. All vehicles assigned to any one traffic lane in loops 2 through 6 had the same axle arrangement - axle load combinations, as described in the AASHO Road Test Axle Configuration Figure below. Tire pressures and steering axle loads were representative of normal practice for the time. The test was conducted over a two-year period, which was probably too short for the complete evaluation of environmental and aging effects, but was sufficient to allow the application of 1,114,000 load applications to each loop. See the following figure for axle weights and distributions used on various loops of the AASHO Road Test.

AASHO Road Test Axle Configuration

Performance measurements were taken at regular intervals to provide information concerning the roughness and visible deterioration over time of the surfacing of each section. These measurements included transverse pavement profiles (rutting), cracking, patching, deflections, strains, layer thicknesses and temperatures, and numerous other measurements. This information

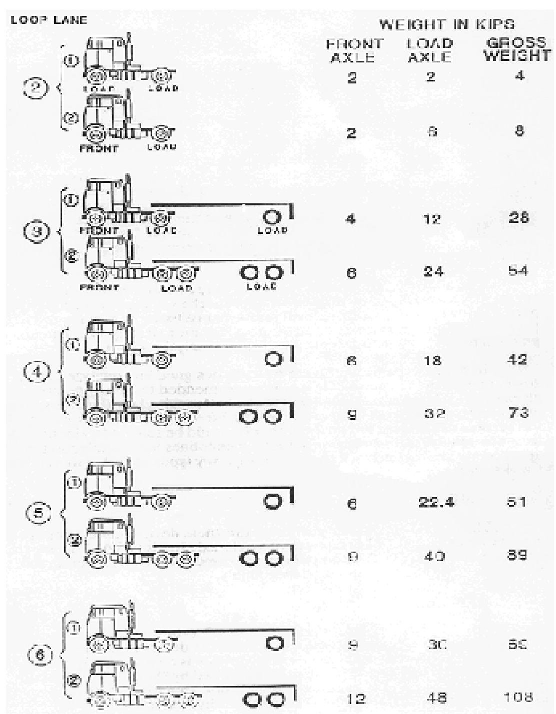


Figure 3D-2

was used in in the development of the performance models that eventually became the basis for the current AASHTO design procedure.

Thickness Design Procedure

Design Inputs

This section describes the design inputs that are required to design flexible pavements. These inputs are classified under four separate categories, as described herein.

General Design Variables

General design variables are those that must be considered in the design and construction of any pavement surface. Included in this category are time considerations (such as selected performance and analysis periods), traffic, design reliability, and environmental effects (such as roadbed swelling and frost heave).

Time Considerations

The performance period is the period of time that elapses as a new or rehabilitated pavement structure deteriorates from its initial serviceability to its terminal serviceability and requires rehabilitation or reconstruction. The designer must select minimum and maximum allowable bounds. The selection of these values is impacted by such factors as pavement functional classification, the public's perception of how long a "new" surface should last, funds available for initial construction, life-cycle costs, and other engineering considerations.

The analysis period is the period of time that any design strategy must cover. The analysis period may be identical to the selected performance period. However, realistic practical performance limitations for some pavement designs may necessitate the consideration of stage construction or planned rehabilitation to achieve the desired analysis period. UDOT does not consider stage construction or planned rehabilitation due to the uncertainty of future funding for these strategies.

The Regional Pavement Management Teams determine pavement design life by incorporating knowledge of planned future corridor projects, future widening and overlay projects, safety projects as well as information from the other management systems besides pavement management, while still balancing the design and capacity needs with funding. The design life used for the pavement design must be verified before beginning the design in the initial consultant design-concept meeting with the UDOT Regional Pavement Management Engineer.

Traffic

The AASHTO flexible pavement thickness design procedures are based on <u>cumulative</u> expected 18-kip equivalent single-axle loads (ESALs) during the analysis period (W_{18}). The computation of cumulative ESALs is described in the "Traffic" section of this "Manual" and should be studied thoroughly. The method for obtaining design ESALs used for the pavement design must be verified before beginning the design in the initial consultant design-concept meeting with the UDOT Regional Pavement Management Engineer.

Reliability

Design reliability refers to the degree of certainty that a given design alternative will last the analysis period. The AASHTO design-performance reliability is controlled through the use of a design reliability factor (F_r) that is multiplied by the design period traffic prediction (w_{18}) to produce design traffic applications (W_{18}) for use in the design equation. For a given reliability level (R), the reliability factor is a function of the overall standard deviation (S_o) that accounts for standard variation in materials and construction, the chance variation in the traffic prediction, and the normal variation in pavement performance for a given W_{18}.

UDOT uses a reliability of 95% for interstates and a reliability of 90% for all other roads.

The selected standard deviation must be representative of local conditions. UDOT uses a value of 0.45 for flexible pavements.

Environmental Impacts

Temperature and moisture changes have an effect on the strength, durability, and load-carrying capacity of the pavement and roadbed materials through the mechanics of swelling soils, frost heave, and other phenomena. Frost action includes both frost heave and the loss of subgrade support during the frost melt period. A discussion of frost mechanics and frost hazard identification is in the "Evaluation Of Site Conditions" section of this "Manual". The effects of frost action may be held to tolerable limits by many single treatments or a combination of several treatment alternatives. Some of these methods include:

1. Partial Removal of Frost-Susceptible Material. This method is commonly used in combination with better drainage.

2. Limit Water Supply by methods such as paved shoulders, side ditching, edge drains, and subdrains. An attempt to improve drainage is almost essential in controlling the damage caused by frost action.

3. Cover Subgrade With Sufficient Depth of Non-Frost-Susceptible Material. This method is commonly used in combination with better drainage.

Removal or covering should be to a minimum of 70% of the predicted frost depth. The predicted frost depth needs to come from the Max. Frost Penetration Depth Map found in the "Site Conditions" section of this "Manual" (figure 3A-4). The UDOT Regional Pavement Management Engineer shall approve the selected frost depth. A copy of the frost depth map, with the project location marked on it, must be submitted as part of the design document.

Performance Criteria

The serviceability of a pavement is defined as its ability to serve the type of traffic that uses the facility. The primary measure of serviceability used by the AASHTO procedures is the Present Serviceability Index (PSI), which ranges from 0 (impassible) to 5 (perfect).

Initial and terminal serviceability indices must be established to compute the total change in serviceability that will be input to the design equations. Initial serviceability index (p_o) is a function of pavement design and construction quality. Typical values from the AASHO Road Test were 4.2 for flexible pavements which is what UDOT uses. Terminal serviceability index (p_t) is the lowest index that will be tolerated before rehabilitation, resurfacing or reconstruction becomes necessary. UDOT uses a terminal serviceability index (p_t) of 2.5 for all roads.

The input required to the AASHTO flexible pavement thickness design procedure is $\Delta PSI = p_o - p_t$. The input for many of the computer design procedures requires both the value for initial serviceability index as well as the value for the terminal serviceability index.

Determination of Materials Properties for Structural Design

The basis for materials characterization in the AASHTO Design Guide[5] is elastic or resilient modulus. The use of these properties in determining flexible pavement design inputs is described below.

Effective Roadbed Soil Resilient Modulus - The AASHTO flexible pavement design procedure requires the input of an effective roadbed soil resilient modulus, which is equivalent to the combined effect of all seasonal modulus values. UDOT uses the CBR value of the soil to determine the effective roadbed soil resilient modulus (M_r) or modulus of subgrade reaction. The equation to convert the CBR value to resilient modulus that UDOT uses is M_r = (CBR value X 1,500). The maximum value that UDOT allows for effective roadbed soil resilient modulus for unstabilized natural soils is 15,000 psi (which corresponds to a maximum allowable CBR value of 10.0).

Pavement Layer Materials Characterization

Although the concept of layer coefficients is still central to the AASHTO flexible pavement design procedure, the 1993 AASHTO Design Guide relies more heavily on the determination of materials properties for the estimation of appropriate layer coefficient values. The preferred tests are the resilient modulus (AASHTO Method T274) for subbase and unbound granular materials and elastic modulus (ASTM D4123 or ASTM C469) for asphalt concrete and other stabilized materials. UDOT uses the following values of resilient modulus based on material properties found in Utah:

Resilient Modulus Values in Utah

Material	Resilient Modulus (M_r) in psi
Untreated Base Course (UTBC)	27,000
Granular Borrow (GB)(A-1-a nonplastic)	15,000

Table 3D-1

Layer Coefficients

The AASHTO flexible pavement layer coefficient (a_i) is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. For example, two inches of a material with a layer coefficient of 0.2 is assumed to provide the same structural contribution as one inch of a material with a layer coefficient of 0.40.

The determination of appropriate layer coefficients can be accomplished by deriving them from test roads or satellite sections (as was done at the AASHTO Road Test) or by using predetermined relationships based on materials properties such as resilient or elastic modulus.

UDOT uses the following values for layer coefficients:

Flexible Pavement Layer Coefficients

Layer	Coefficient (per inch)
Plant Mix Seal (PMS)	0.30
Hot Mix Asphalt (HMA)	0.40
Untreated Base Course(UTBC)	0.10
Granular Borrow (GB)	0.08

Table 3D-2

Pavement Structural Characteristics

Drainage

The AASHTO flexible pavement design procedure provides a means to adjust layer coefficients to take into account the effects of certain levels of drainage on pavement performance. Guidance concerning the design or effectiveness of various drainage approaches is not provided; the design engineer must identify the level or quality of drainage that is achieved under a specific set of drainage conditions. The following is a guide to qualify drainage:

Drainage Quality Classification

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	water will not drain

Table 3D-3

The effect of drainage of all untreated layers below the surface is considered by multiplying the layer coefficients, a_i, by a modifying factor (m_i). This factor can be obtained from the following table and is a function of the drainage characteristics of the roadbed soil (as categorized above) and the amount of time the soil is in a saturated condition:

Drainage Coefficient Values

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	< 1%	1% - 5%	5% - 25%	> 25%
Excellent	1.4 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.05 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Table 3D-4

The structural number equation modified for drainage becomes:

$$\mathrm{SN} = a_1D_1\%a_2D_2m_2\%a_3D_3m_3$$

where:

a_i =layer coefficient of layer I
D_i = thickness of layer I, inches
m_i = drainage modifying factor for layer

The drainage conditions at the AASHO Road Test are assumed to be "fair" and the m_i values there are assumed to be 1.0, regardless of the material. It should be noted that these values are assumed because the structural models should not require adjustment for the conditions at the Road Test. However, these same materials would probably receive drainage modifying factors of less than 1.0 for a new construction project, and the designer should select appropriate values to reduce the possibility of a poor design.

The maximum value that UDOT accepts for the drainage modifying factor is 1.0. This corresponds to the unshaded portion of the chart above. The designer must verify the correct value to use with the UDOT Regional Pavement Management Engineer in the initial consultant design-concept meeting.

The drainage modifying factor applies only to the effects of drainage on untreated base and subbase layers.

Stability and Constructability

It is generally impractical and uneconomical to place layers of material that are less than some minimum thickness. The following values are UDOT's standard thicknesses for surface and base layers:

UDOT Standard Layer Thicknesses

Plant Mix Seal (PMS)	1"
Hot Mix Asphalt (HMA) Total	2.5" min.
Hot Mix Asphalt (HMA) Lift	2" min., 4" max.
Untreated Base Course (UTBC)	4" min.
Granular Borrow (GB)	6" min.

Table 3D-5

The thicknesses of the layers should also be balanced (the thickness of a layer should be greater than the thickness of the layer immediately above it).

Computation of Required Pavement Thickness

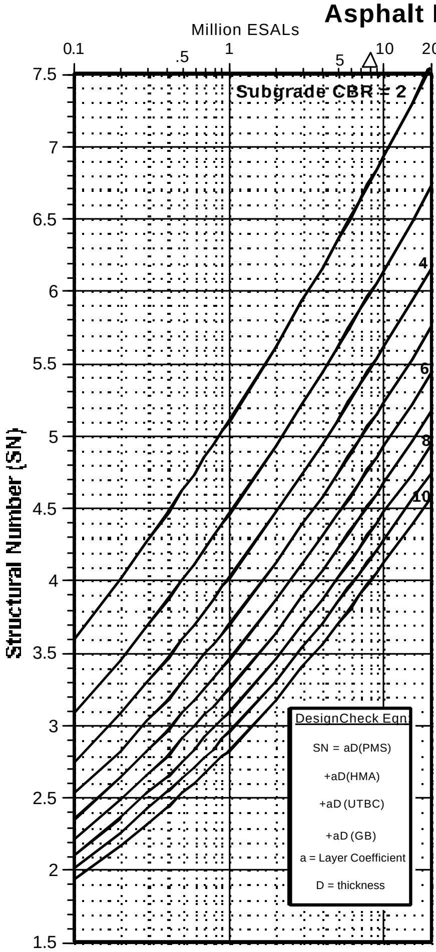
The AASHTO flexible pavement design process can be accomplished using the design inputs and design equations already presented. These equations can be solved manually, using a series of nomographs, using UDOT's developed design charts, and using computer software. The complexity of the design procedure can make the manual solution a tedious process. Although the nomographs simplify the process, they include some inherent assumptions that make their solutions somewhat less precise than those provided by the manual, UDOT design chart, and computer solutions. UDOT will not accept a design submitted using nomographs. UDOT's design charts simplify the design process even further by incorporating UDOT's design variables into the charts. They are valuable in that they give a visual indication of sensitivity. The computerized approach allows easy consideration of all design factors and provides accurate solutions to the design equations.

The use of UDOT's design charts will be explained in this "Manual" since they are the easiest to use and are preferred by UDOT. Even though the use of nomographs is not accepted by UDOT, their use is explained in the appendix at the end of this section since they are referred to in the 1993 AASHTO Design Guide. The use of computer programs is recommended as a check of the UDOT design charts design. Specific computer programs will not be discussed or explained since there is such a variety, and they are constantly changing and improving. If a computer program is used as a check of the UDOT design charts design, it must be an approved computer program. Approval of a computer program must come from the Regional Pavement Management Engineer and should be obtained in the initial consultant design-concept meeting.

UDOT's Design Charts Method of Flexible Pavement Design

There are two sets of graphs. One set is for 95% reliability (Interstates) and the other set is for 90% reliability (Other Than Interstates). Each set of graphs is on only one sheet of paper. See the two sets of graphs below:

Asphalt Pavement Thickness Evaluation (95% reliability)



Initial Servicability Index = 4.2

Terminal Servicability Index = 2.5

Drainage Coefficients = 1.0

Overall Standard Deviation = 0.45

△ Upper Limit of AASHTO Road Test Traffic (8,000,000 ESALs)

Layer Coefficients (a) and Resilient Modulus (M_R)

Hot Mix Asphalt (HMA): a = 0.0158 per mm (0.40 per inch)

Plant Mix Seal (PMS): a = 0.0118 per mm, (0.30 per inch)

Untreated Base (UTBC): a= 0.0039 per mm (0.10 per inch), M_R = 27 ksi (186 MPa)

Granular Borrow (GB): a = 0.0031 per mm (0.08 per inch), M_R = 15 ksi (103 MPa)

Subgrade M_R (ksi) = 1.5 x Saturated CBR, (MPa) = 10.3 x CBR

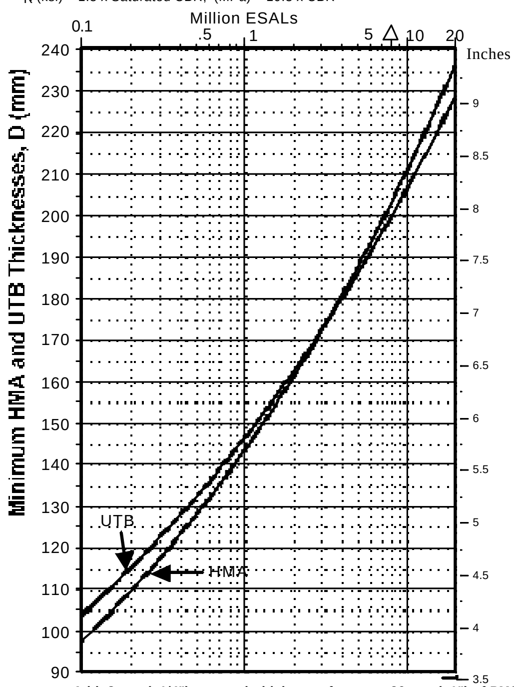
Project:

By: **Date:**

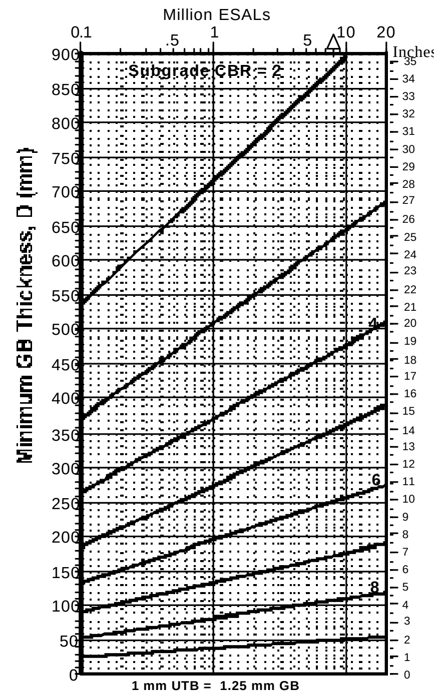
NOTE **A:**

NOTE **B:**

NOTE **C:**



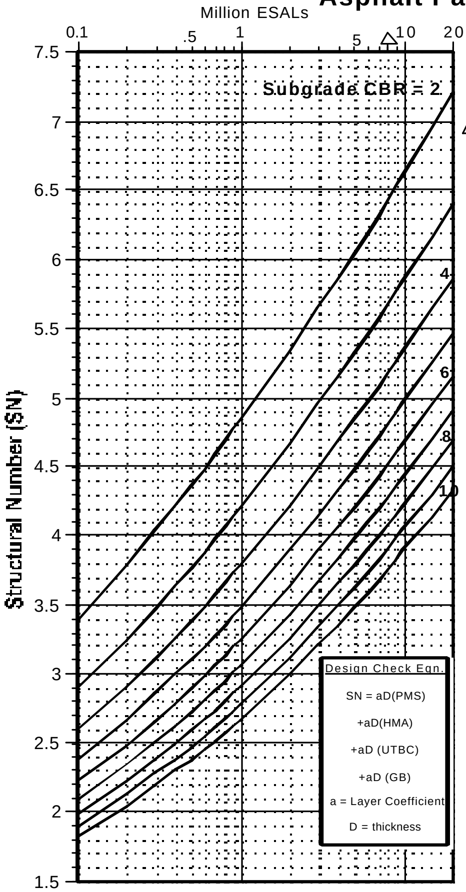
Add 8 mm (+1/4") to graph thickness for every 30 mm (+1") of PMS



1 mm UTB = 1.25 mm GB

Graphs Developed Using DARWin 2.0

Asphalt Pavement Thickness Evaluation (90% reliability)



Initial Servicability Index = 4.2

Terminal Servicability Index = 2.5

Drainage Coefficients = 1.0

Overall Standard Deviation = 0.45

△ Upper Limit of AASHTO Road Test Traffic (8,000,000 ESALs)

Layer Coefficients (a) and Resilient Modulus (M_R)

Hot Mix Asphalt (HMA): a = 0.0158 per mm (0.40 per inch)

Plant Mix Seal (PMS): a = 0.0118 per mm, (0.30 per inch)

Untreated Base (UTBC): a = 0.0039 per mm (0.10 per inch), M_R = 27 ksi (186 MPa)

Granular Borrow (GB): a = 0.0031 per mm (0.08 per inch), M_R = 15 ksi (103 MPa)

Subgrade M_R: (ksi) = 1.5 x Saturated CBR, (MPa) = 10.3 x CBR

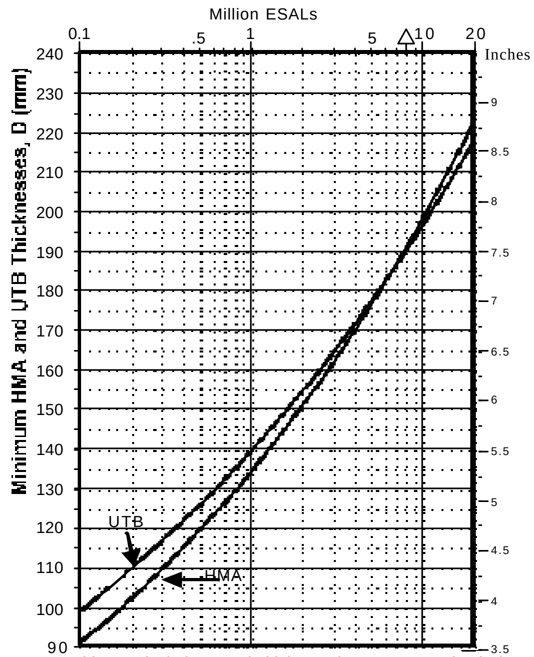
Project:

By: **Date:**

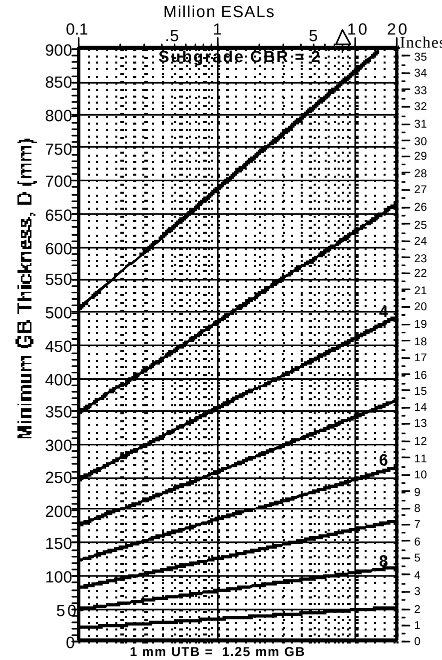
NOTE **A:**

NOTE **B:**

NOTE **C:**



Add 8 mm (+1/4") to graph thickness for every 30 mm (+1") of PMS



1 mm UTB = 1.25 mm GB

Graphs Developed Using DARWin 2.0

Roadbed Soil Structural Number

Once the correct sheet of paper with the correct set of graphs has been selected, use the Structural Number graph on this sheet to determine the required structural number (SN) above the roadbed soil. Using the estimated traffic for the performance period (ESALs), go down the chart to the intersection of the curve of the CBR value of the roadbed soil you are building on. Draw a horizontal line where these two values intersect and read across to determine the required SN. Substitute this value into the "Design Check" equation found on this same sheet of paper. This SN will be used later (as explained in the Design Check section) to check against the summation of the structural numbers of the individual pavement layers.

Hot Mix Asphalt and Untreated Base Layers

Use the Minimum Asphalt and UTB Thicknesses chart to determine the required thickness of the hot mix asphalt (HMA) layer and the required thickness of the untreated base course (UTBC) layer. As done previously, use the ESALs to go down the chart and intersect the curves for HMA and UTBC. Draw horizontal lines where these intersections occur and read across to determine the required thicknesses of these two layers. Round up to the next 10.0 mm for each of these layers. If a one inch thick plant mix seal coat (PMSC) will be placed on the hot mix asphalt, then the thickness of HMA determined from the chart as explained above needs to be reduced by 20 mm. Make sure the HMA layer thickness is greater than or equal to the minimum layer thickness required for construction (2.5 inches). Make sure the UTBC layer thickness is greater than or equal to the minimum layer thickness required for construction (4 inches), and check to make sure the layer thickness is greater than the thickness of the HMA layer above it.

Granular Borrow Layer

Use the Minimum GB Thickness chart to determine the required thickness of the granular borrow (GB) layer. Again, read down the chart with the ESALs to the intersection of the curve of the CBR value of the roadbed soil you are building on. Draw a horizontal line where these two values intersect and read across to determine the required thickness of GB. Round this value up to the nearest 10.0 mm. Make sure the thickness of the GB layer is greater than or equal to the minimum layer thickness required for construction (6 inches), and check to make sure the layer thickness is greater than the thickness of the UTBC layer above it.

Design Check

Substitute all of the selected layer thicknesses into the "Design Check" equation along with their corresponding structural coefficients. Check to make sure that the summation of the layer structural numbers is greater than or equal to the structural number of the roadbed soil.

Check to make sure the layer thicknesses satisfy UDOT's minimum layer thicknesses requirements.

Check to make sure the layer thicknesses are balanced (the thickness of a layer should be greater than the thickness of the layer immediately above it).

Check to make sure the total depth of the pavement structure satisfies UDOT's minimum depth requirements for frost protection (70% of the predicted frost depth). Refer to the Maximum Frost Penetration Depth Map presented earlier.

Life Cycle Cost Analysis

A life cycle cost analysis shall be performed to evaluate all design alternatives. This is explained and discussed in the "Life Cycle Cost Analysis" section of this "Manual".

Limitations of the AASHTO Design Procedure

Major limitations of the AASHTO flexible pavement design procedure are summarized as follows:

1. ***Limited materials and subgrade***. The Road Test used a specific set of pavement materials and one roadbed soil. The extrapolation of the performance of these specific materials to general applications is dangerous because the materials and soils available locally will probably not be identical to those used at the Road Test and will perform differently. The AASHTO design procedure addresses this deficiency through the use of several adjusting factors and inputs (M_R, F_R, drainage, etc.). However, many of these inputs are also based on empirical relationships and must be used carefully.

2. ***No mixed traffic***. The AASHO Road Test accumulated traffic on each test section by operating vehicles with identical axle loads and axle configurations. In-service pavements are exposed to many different axle configurations and loads. The process of converting mixed traffic into equivalent 18-kip ESAL applications is based on another empirical relationship that has never been field-verified.

3. ***Short road test performance period***. The number of years and heavy axle load applications upon which the design procedure is based represents only a fraction of the design age and load applications that many of today's pavements must endure. Even if the AASHTO equations can be extrapolated to design for many load applications, the environmental deterioration that occurs over time is not directly included in the design equations.

4. ***Load equivalency factors***. The load equivalency factors used to determine cumulative 18-kip ESALs pertain specifically to the Road Test materials, pavement composition, climate and subgrade soils. The accuracy of extrapolating them to other regions, materials and distresses, etc., is not known, but is questionable.

5. ***Variability***. A serious limitation of the AASHTO design procedure is that it is based upon very short pavement sections where construction and material quality were highly controlled. Typical highway projects are normally several miles in length, and contain much greater construction and material variability, and hence show more variability in performance along the project in the form of localized failures. Projects designed using average inputs could be expected to exhibit significant localized failures before the average project serviceability index drops to p_t, unless a level of reliability somewhat greater than that desired is selected for design.

6. ***Lack of guidance on some design inputs***. Structural coefficients and drainage modifying factors are very significant on influencing flexible pavement layer thicknesses, and there is very little guidance provided for their selection. The design reliability also has an extremely large effect on pavement thickness and very little guidance is provided in selecting this factor.

Successful use of the AASHTO Guide requires a lot of experience and knowledge of the assumptions and underlying basis for design. It is strongly recommended that the resulting designs be checked using other procedures and mechanistic analyses.

Summary

The AASHTO Road Test was a carefully designed and constructed experiment that provided data concerning the effects of varying thicknesses of surfacing, base course and subbase. Other design factors were also studied. These data were used to develop empirical performance prediction models, which formed the basis for a pavement design procedure.

Over the years, several modifications and improvements have been made to the AASHTO procedures, with the most significant changes occurring in 1986. The current design procedure incorporates the concepts of design reliability and variability of design inputs, the use of resilient modulus testing for materials characterizations and determination of layer coefficients, direct consideration of the effects of drainage on materials performance, and a rational approach to adjusting designs to account for environmental considerations.

The current AASHTO flexible pavement design procedure can be accomplished manually, through the use of a nomograph, use of UDOT design charts, and use of computer programs. UDOT will only accept designs that use UDOT design charts and approved computer programs. UDOT will not accept designs that use nomographs. Design inputs include performance and analysis periods, cumulative expected traffic in ESALs, required design reliability, performance criteria (initial and terminal serviceability), and resilient or elastic modulus, and drainage characteristics of the paving materials and roadbed soil. Trial thicknesses are adjusted to minimize initial construction costs and to provide layer stability and construct ability.

Most of the major limitations of the AASHTO flexible pavement design procedure are concerned with the general use of a design procedure that was developed from very specific conditions (e.g., one climate, one set of materials, no mixed traffic, etc.) over a short period of time. The successful use of the AASHTO design procedure requires experience and a knowledge of the underlying assumptions. AASHTO designs should be verified using other design procedures and mechanistic analyses.

References

1. , "AASHO Interim Guide for the Design of Rigid Pavement Structures," American Association of State Highway Officials, Committee on Design, April 1962.

2. , "AASHO Interim Guide for Design of Pavement Structures, "American Association of State Highway Officials, Washington D.C., 1972.

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Appendix 3D

AASHTO's Design Nomograph Method of Flexible Pavement Design

The basic AASHTO design process for flexible pavements begins with the determination of required structural number (SN) based on the level of traffic. Trial pavement designs are then identified by using different layer thicknesses that provide the required structural number, meet minimum layer thickness criteria, and provide adequate protection for underlying materials.

Determination of the Required Structural Number

The following figure presents the recommended nomograph for determining the design structural number (SN) required for specific conditions:

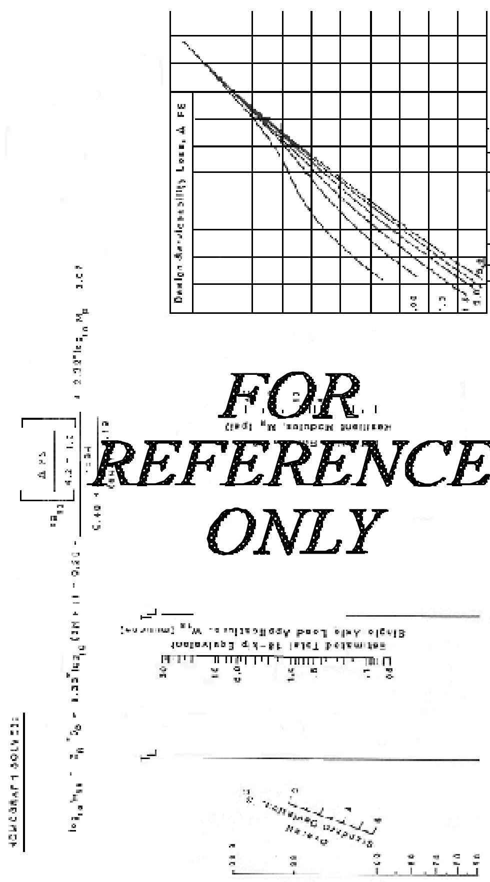


Figure C-1.5 AASHTO Design Nomograph

This nomograph solves the following equation:

$$\log_{10}W_{18} = Z_R S_O + 9.36\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.04 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log_{10}M_R - 8.07$$

The required inputs are:

1. The estimated future traffic, W_{18}, for the performance period.
2. The reliability, R, which assumes that average values are used for all inputs.
3. The overall standard deviation, S_O.
4. The effective roadbed soil resilient modulus, M_R.
5. The design serviceability loss, $\Delta PSI = p_O - p_t$.

Once the design structural number for an initial pavement structure has been determined, the designer must identify a set of pavement layer thicknesses which will provide the required load-carrying capacity that corresponds to the design structural number. The following equation, which was presented earlier, provides the means for converting the structural number into actual thicknesses of surfacing, base and subbase materials:

$$\text{SN} = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

where:

a_i = layer coefficient of layer I

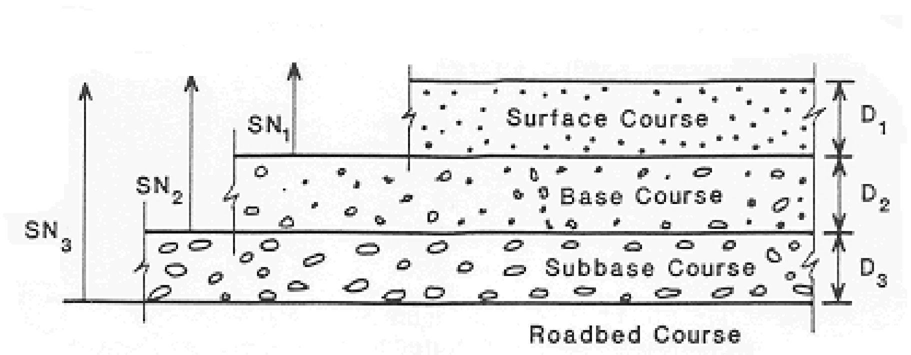
D_i = thickness of layer I, inches

m_i = drainage modifying factor for layer I

This equation does not have a single unique solution. There are many combinations of layer thicknesses that can be used to achieve a given structural number. There are, however, several design, construction and cost constraints that can be applied to reduce the number of possible layer thickness combinations and to avoid the possibility of constructing an impractical design. Some of these constraints are presented below.

Layered Design Analysis

Flexible pavement structures are layered systems and should be designed accordingly. Each unbound or aggregate layer must be protected from excessive vertical stresses that could result in permanent deformation. This requires that minimum thickness values be observed depending on the traffic requirements. The procedure to accomplish this is



shown in the following Layer Thickness Figure and formulas:

$$D_1^* \geq SN_1/a_1 = D_1$$

$$SN_1^* = a_1 D_1^* \geq SN_1$$

$$D_2^* \geq (SN_2 - SN_1^*)/a_2 m_2$$

$$SN_1^* + SN_2^* = a_1 D_1^* + a_2 m_2 D_2^* \geq SN_2$$

$$D_3^* \geq [SN_3 - (SN_1^* + SN_2^*)]/a_3 m_3$$

$$SN_1^* + SN_2^* + SN_3^* = a_1 D_1^* + a_2 m_2 D_2^* + a_3 m_3 D_3^* \geq SN_3$$

Figure C-1.6 Layer Thickness Figure And Formulas

The design nomograph previously presented can be used to determine the design structural numbers required for the protection of <u>any</u> unbound layer by substituting the resilient modulus of <u>that</u> layer for the roadbed resilient modulus in the nomograph. The minimum required surface course thickness, D_1, is then determined by dividing the structural number required to protect the base course by the layer coefficient of the surface course (SN_1/a_1). The selected surface course thickness, D^{*}_1, must be greater than or equal to this minimum thickness.

$$D^{*}_1 \geq SN_1/a_1 = D_1$$

The actual structural number provided by the selected surface course thickness for the protection of the base course is computed as:

$$SN^{*}_1 = a_1 D^{*}_1 \geq SN_1$$

The minimum required base course thickness is determined in a similar manner. The structural number provided by the thickness of surface provided ($a_1 D^{*}_1 = SN^{*}_1$) is subtracted from the structural number required to protect the subbase course, SN_2. This quantity is divided by the product of the base course layer coefficient and drainage modifying factor to determine the minimum required base course thickness, D_2. The selected base course thickness, D^{*}_2, must be greater than D_2.

$$D^{*}_2 \geq (SN_2 - SN^{*}_1)/a_2 m_2$$

The actual structural number provided by the surface and base course for the protection of the subbase course is computed as:

$$SN^{*}_1 + SN^{*}_2 = a_1 D^{*}_1 + a_2 m_2 D^{*}_2 \geq SN_2$$

Finally, the minimum required subbase thickness is determined. The structural number provided by the selected surface and base course thicknesses (SN^{*}_2) is subtracted from the structural number required to protect the roadbed soil, SN_3. This quantity is divided by the product of the subbase layer coefficient and drainage modifying factor to determine the minimum required subbase thickness, D_3. The selected base course thickness, D^{*}_3, must be greater than D_3.

$$D^{*}_3 \geq [SN_3 - (SN^{*}_1 + SN^{*}_2)]/a_3 m_3$$

The actual structural number provided by the three upper layers for the protection of the roadbed soil is computed as:

$$SN^{*}_1 + SN^{*}_2 + SN^{*}_3 = a_1 D^{*}_1 + a_2 m_2 D^{*}_2 + a_3 m_3 D^{*}_3 \geq SN_3$$

This approach provides trial designs that meet the structural requirements for the protection of each layer in the pavement structure. This procedure should not be applied to determine the required layer thickness above materials having a resilient modulus of greater than 40,000 psi. Layer thickness above such "high modulus materials" should be established based on cost-effectiveness and minimum practical thickness considerations.

Section 3E
Life Cycle Cost Analysis
For Pavement Design and Rehabilitation

(Adapted from the value engineering manual, February 23, 1994)

Introduction

Engineering economics provides a way to choose between alternatives when the expenditure of limited funds is a factor. It also allows budgetary and environmental impacts to be considered. The format and framework for the analysis must be decided at the initial concept meeting. This applies to all projects, UDOT or local government.

There are three steps to consider in this process. First, identify and define the different alternatives. Second, identify and define the factors that may result in cost differences. Third, eliminate all elements which may happen regardless of which alternative is selected.

Life cycle cost (LCC) analysis is the most appropriate economic evaluation process in deciding between alternatives. This analysis considers the cost of construction, rehabilitation, maintenance, and associated user impacts over a specific period. This period usually encompasses the service life of all the alternatives.

There are two important definitions to know. Life cycle costing —"Economic assessment of an item, area, system, or facility and competing design alternatives considering all significant costs of ownership over the economic life, expressed in terms of equivalent dollars"[6] and Life cycle design—"Analysis which considers the construction, operation, and maintenance of a facility during its entire design life."[7]

In general, life cycle costs include all costs anticipated over the life of the facility. As part of the analysis, trade-offs can be made among factors that may affect the life cycle cost of a pavement, such as the relationship between the initial costs of construction and the future cost of maintenance. The analysis requires identifying and evaluating the economic consequences of various alternatives over time or the life cycle of the alternative.[8]

Costs normally associated with pavement reconstruction include:

- Initial construction costs
- Maintenance costs
- Rehabilitation costs
- User costs

UDOT does not consider salvage or energy costs in the LCC analysis. LCC analysis, the availability of funds, project specific and environmental conditions or constraints, and constructability should be used in the decision process for selecting the most appropriate alternative.

Engineering Economics

There are three concepts that form the basis of life cycle analysis methodology.

1. Time value of money
2. Discount rate
3. Analysis period

Time Value of Money

Two factors influence the time value of money. They are rate of return and inflation. The rate of return is the amount of money earned from the use of capital. The rate is calculated for a specific investment. The complexity of the determination varies depending on the length of time considered for the investment. Inflation is a general increase in the level of prices throughout the economy. Highway agencies do not normally include inflation when analyzing alternatives because of the uncertainty in predicting future inflation rates. Because only differential inflation on future costs requires identification, the annualized dollar method is used. This method brings all costs, present and future, to present dollars.

Discount Rate

Use the discount rate as a means to compare alternative uses of funds by reducing the future expected costs or benefits to present day terms. Discount rates reduce costs or benefits to their present worth or annualized costs. The economics of the alternatives can then be compared. The discount rate will be established by the UDOT value engineering group. Contact the Regional pavement management engineer for the current rate.

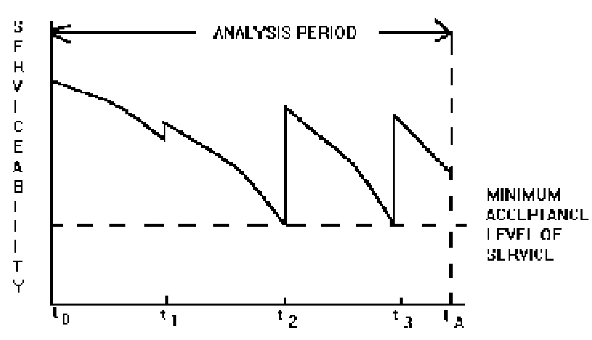


Figure 19 Performance Curve

Analysis Period

The final component that should be established before performing an LCC analysis is to select an appropriate comparison time period. The analysis period is the total length of time the facility is expected to serve its intended function or the time frame before the component in question requires replacement or upgrade. This period may contain several maintenance and rehabilitation activities. Figure 1 illustrates an example of these activities for pavement performance.

The recommended analysis period for each project shall be determined by the Regional pavement management team. For local government projects the actual analysis period for project specific designs shall be decided at the initial concept meeting.

Discounted Cash Flow Analysis

The only analysis option allowed by UDOT is the annualized method.

Annualized Method

This method converts present and future expenditures to a uniform annual cost, resulting in a common base of a uniform annual cost. This method divides expected costs over the life of the project into uniform annual costs using the appropriate discount rate. This analysis must be done for initial, recurring, and nonrecurring costs. Maintenance expenditures are recurring annual costs and must be estimated and included. The future costs need to be estimated in present day dollars. The following equation may be used to determine the annualized cost.

$$A = P\{[I(1 + I)^n]/[(1 + I)^n - 1]\}$$

where P = Present worth
A = Annualized cost or annual cost
n = Number of years
I = Discount rate

The factor $[I(1 + I)^n]/[(1 + I)^n - 1]$ is also know as the Uniform Capital Recovery Factor (UCR).

or

(A/P,i%,n)

A simplified calculation for A involves multiplying P by the UCR.

Sensitivity Analysis

Sensitivity is the relative effect that each variable may have on the choice of alternatives. A sensitivity analysis tests the effects of the variations. This analysis may be used at the discretion of the Regional pavement management engineer or should be considered if two options indicate relatively close life cycle costs.

Cost Factors

Initial Costs

The most common reconstruction or rehabilitation alternatives are remove and replace with new Portland Concrete Cement Pavement (PCCP), remove and replace with new Asphalt Concrete Pavement (ACP), or overlay with either ACP or PCCP. The Regional pavement management engineer coordinates all details regarding the design of alternatives.

Reflect all initial construction costs, unique or regular, associated with each alternative.

Maintenance Costs

These costs are those associated with maintaining the pavement surface at some acceptable level and are one of the most difficult areas to deal with in LCC analysis. The differential in maintenance requirements for the various alternatives being considered is the most important item. If maintenance costs are identical for all alternatives, then there would be no need to include maintenance in the analysis.

Rehabilitation Costs

These costs are associated with pavement rehabilitation or restoration activities. Typical costs include chip seal coat, fabric interlayers, asphalt-rubber interlayers, open-graded friction course, and dense-graded asphalt concrete. The costs of traffic control need to be included unless the traffic control plan is identical in all alternatives.

These costs are computed in the same manner as initial construction costs. With respect to pavement rehabilitation, projects are normally bid and constructed under the same criteria as new pavement construction. When considering the costs of rehabilitation relative to LCC analysis, two time frames come into play. The first time frame applicable to many projects begins at "time zero." This constitutes the beginning of an LCC analysis and applies where the pavement existed for years. In this case, treat rehabilitation similar to initial construction. The second time frame applies to future needs for a new pavement or a newly rehabilitated pavement. A guideline for these needs is the UDOT Pavement Life Strategy dated November 1998. This document is included in the Appendix of this section.

User Costs

These costs are those associated with vehicle operating costs such as fuel consumption, parts, tires, etc. There are user delay costs such as denial-of-use, speed change delays, and idling time. The parameters to be used shall be defined at the initial concept meeting. This is an informal analysis. It's use shall be determined by the Regional pavement management engineer and the Regional pavement management team.

The following equation may be used to determine user costs.

UC=(AVT)[(L/RS) - (L/IS)](ADT)(PT)(CP)

where UC = User Cost (total project costs)
AVT = Average Value of Time (Use values from the table below (1996 dollars) or a value determined by a more detailed analysis)

Vehicle Class	$/ Vehicle Hour Range
Passenger Vehicles	10.00 - 13.00
Single Unit Trucks	17.00 - 20.00
Combination Trucks	21.00 - 24.00

L = Project Length
RS = Reduced speed through construction zone
IS = Initial speed prior to construction zone
ADT = Average daily traffic in current year (only portion of ADT affected by the project
PT = Percent of the traffic affected by the construction project. Perform traffic study to determine percent of traffic using facility during the period.
CP = Construction period in days

Procedures

Generating Alternative Designs

The previous sections discussed cost factors and the economics of life cycle cost analysis. This section puts all that together, explaining the process to complete the analysis. It must first be determined whether to reconstruct or rehabilitate. Then identify all alternatives meeting design requirements and unique site characteristics. The pavement condition drives the rehabilitation alternatives.

After identifying all possible alternatives, a selection of the most promising choices must be made. The following criteria should be used in making this evaluation.

1. Will the idea work?
2. What is the life cycle cost savings potential?
3. Can it be built?
4. Will it satisfy the need's?
5. Does it match the corridor studies for the route in question?
6. Does it consider the Regional pavement team preferences?

All alternatives should receive a preliminary evaluation before elimination. A list of advantages and disadvantages for each alternative helps to narrow the list. Develop the remaining alternatives with more detail designs and cost estimates. The remaining alternatives must adequately perform the required objectives.

Design Evaluation/ Life Cycle Costing

The next step is to reduce the remaining alternatives by comparing and ranking the ideas within each alternative. Judge advantages or disadvantages objectively. Include in this process an estimate of the potential reduction in life cycle costs and how each alternative meets required functions. Develop the remaining alternatives, including more detailed cost estimates. Be as accurate and consistent as possible between alternatives. With the preliminary evaluation complete and the best available cost data on hand, the remaining alternatives are ready for LCC analysis. All costs must be figured using the annualized method. This analysis should follow the procedures listed in this manual. Remember that the baseline analysis for pavement rehabilitation is reconstruction. This is an alternative that must always be evaluated.

Design Selection

The final step is to recommend an alternative. The life cycle cost analysis will indicate a specific alternative. There are other factors to consider in the final decision. These include, but are not limited to, the following : safety, similar pavement performance, weather cycles, highway system needs, construction constraints and local government preference if the project is not for DOT.

Recommendations

The following procedural steps apply when selecting between alternatives for new or rehabilitated pavements.

1. Determine the site characteristic and other input data that may influence the pavement design or the rehabilitation design. For rehabilitation projects include a detailed pavement evaluation.

2. Identify various pavement management strategies that might be used to achieve the requirements for the project. The baseline assumption is reconstruction.

3. In all situations, identity all feasible alternatives satisfying project needs. Alternatives should provide the required structural service life for the analysis period.

4. Remove from further consideration any item or items common to all alternatives.

5. Select the analysis period. This will vary from rehabilitation to reconstruction to new construction. This will be determined by the Regional pavement management team.

6. Use the discount rate provided by the UDOT value engineering section. Contact the Regional pavement management engineer for the current rate.

7. Use the UDOT design life strategy (Appendix 3E) to identify time intervals for future maintenance and rehabilitation activities.

8. Estimate costs for each alternative in present dollars.

9. Calculate the annualized costs for each alternative.

10. Items or factors subject to variation may require a sensitivity analysis to ensure selection of the proper alternative.

11. Evaluate alternatives against potentially overriding factors.

12. Considering all evaluated factors, select the most promising or preferred new pavement or rehabilitation design.

Summary of Credits

The majority of this LCC procedure is based on the “Life Cycle Cost Analysis of Pavements” report complied by Dale E. Peterson for the National Cooperative Highway Research Program Synthesis of Highway Practice for the Highway Research Board, National Research Council. Most of the references used in this procedure were taken from Peterson’s report.

References

1. Dell'Isola, A.J. and S.J. Kirk, *Life Cycle Costing for Design Professionals*, McGraw-Hill, New York, 1981.

2. Lindow, E.S., *Systems Approach to Life-Cycle Design of Pavements*, Vol I, *Life 2 Users Manual*, Report No. CERL-TR-M-253, Construction Engineering Research Laboratory, Dept of Army, 1978.

3. Peterson, Dale E., National Cooperative Highway Research Program Synthesis of Highway Practice No. 122: "Life-Cycle Cost Analysis of Pavements," Highway Research Board, National Research Council, Washington, D.C., 1985.

Appendix 3E

Pavement Life Strategy

(UDOT Pavement Management Team, November 1998)

The purpose of this document is to provide guidelines for estimating pavement life cycle treatments and guidelines for estimating funding needs for a pavement preventative maintenance program. Pavement preventative maintenance is a planned strategy of cost effective treatments to an existing roadway system that retards deterioration and improves the functional condition of the system without substantially increasing structural capacity.

Pavement Activities

Typical pavement activities fall in three general categories, maintenance, rehabilitation, construction (or reconstruction). Descriptions of these activities follow.

Maintenance - activities in this area are subdivided into responsive or preventative actions.

Responsive maintenance fixes defects as they occur. It is reactive. Responsive activities include skin patching, pothole patching, treating bleeding pavements, spot surface replacement, spot base repair, lane leveling, Portland Cement Concrete (PCC) crack sealing and PCC spot slab replacement.

Preventative maintenance preservation activities are designed to extend the service life of an existing pavement. Preventative maintenance can programed on an recurring basis for specific roads. The work is accomplished by contract or by state forces. The main purpose of this type of maintenance is to try to prevent damage caused by water and other environmental and loading damages. This maintenance includes thin overlays (<=1.5 inches), rotomilling, surface seals (chip seals, open graded seals, slurry seals, and micro surfacing), rejuvenation , lane leveling, crack sealing, concrete joint resealing and spall repair, concrete grinding, slab jacking and/or undersealing and minimal full depth concrete slab replacement.

Rehabilitation - this activity calls for major improvements to the pavement surface that begin a new service life cycle. These improvements are programmed on a project basis. They are based on identified needs, system priorities and available funds. These projects are programmed into the Utah Statewide Transportation Improvement Program (STIP). STIP activities include removal and replacement of more than 1.5 inches of bituminous surface course, removal and recycling more than 1.5 inches of bituminous surface course, structural overlays greater than 1.5 inches, concrete slab replacements (up to 15%), concrete grinding, slab jacking, slab undersealing and dowel bar retrofitting.

Construction/Reconstruction - this activity involves complete facility reconstruction or construction of facilities on new alignment. Like rehabilitation these activities are programmed on a project basis based on identified needs, system priorities and available funds. While new construction is clearly understood; reconstruction includes removal and replacement of the entire pavement using recycled or new material, roadway geometric improvements, safety, improvements, capacity improvements, slab replacement (>15%) and paving of previously unpaved roadways.

Pavement Life Strategy

UDOT has developed the following strategies to preserve the useful pavement life. UDOT pavements are divided into the following five preservation treatment categories:

1: Concrete Pavements on all systems
Asphalt Pavements
- 2: Interstate
- 3: High Volume (AADT>=5,000)
- 4: Low Volume (AADT<5,000)

5: Gravel Roads

The criteria listed above for high volume and low volume asphalt roads is a guide. Roads may be put in either category if there is some engineering justification for varying from the above stated criteria.

The following strategies are guidelines for projecting treatments for life cycle cost analysis and preservation funding. The specific treatments that are performed and the exact timing, should be based upon the actual condition of the individual pavements as determined by inspections. The preservation categories may share treatment types, but the actions may occur at different frequencies.

The starting point for each pavement cycle is initial construction or a new pavement. The other actions are then phased to preserve the surface in a good condition. The actions are shown below and are listed by categories.

Category Number 1: **Concrete Pavements (On All Systems)**

The typical type of treatments and time intervals for this category include:

Treatment	Time Interval - Years
Joint seal, grinding and joint repair	10
Partial and full depth slab repairs	20

A projected treatment life cycle after initial construction would be:

Joint seal, grinding and joint repair.......................... 10^{th} year
Partial and Full Depth Repairs................................. 20^{th} year
Joint seal, grinding and joint repair.......................... 30^{th} year
New pavement structure... 40^{th} year

Category Number 2: **Interstate Asphalt Pavements**

The typical type of treatments and time intervals for this category include:

Treatment	Time Interval - Years
Surface Seal	6 after surface seals and right after overlays
Surface Rejuvenation	3 after surface seals
Structural Overlay and Surface Seal	15

A projected treatment life cycle after initial construction would be:

Surface Rejuvenation... 3^{rd} year
Surface Seal... 6^{th} year
Surface Rejuvenation... 9^{th} year
Surface Seal... 12^{th} year
Structural Overlay and Surface Seal......................... 15^{th} year
Surface Rejuvenation... 18^{th} year
Surface Seal... 21^{st} year
Surface Rejuvenation... 24^{th} year
Surface Seal... 27^{th} year
Structural Overlay and Surface Seal......................... 30^{th} year
Surface Rejuvenation... 33^{rd} year
Surface Seal... 36^{th} year
Surface Rejuvenation... 39^{th} year
Surface Seal... 42^{nd} year
New Pavement Structure.. 48^{th} year

Category Number 3: **High Volume Asphalt Pavements (AADT>=5,000)**

The typical type of treatments and time intervals for this category include:

Treatment	Time Interval - Years
Surface Seal	8 after surface seals and right after overlays
Surface Rejuvenation	3 after surface seals
Structural Overlay and Surface Seal	15

A projected treatment life cycle after initial construction would be:

Surface Rejuvenation... 3rd year
Surface Seal... 8th year
Surface Rejuvenation... 11th year
Structural Overlay and Surface Seal......................... 15th year
Surface Rejuvenation... 18th year
Surface Seal... 23rd year
Surface Rejuvenation... 26th year
Structural Overlay and Surface Seal......................... 30th year
Surface Rejuvenation... 33rd year
Surface Seal... 38th year
Surface Rejuvenation... 41st year
New Pavement Structure... 48th year

Category Number 4: **Low Volume Asphalt Pavements (AADT<5,000)**

The typical type of treatments and time intervals for this category include:

Treatment	Time Interval - Years
Surface Seal	6 after surface seals, and right after overlays
Surface Rejuvenation	3 after surface seals
Structural Overlay and Surface Seal	23

A projected treatment life cycle after initial construction would be:

Surface Rejuvenation... 3rd year
Surface Seal.. 6th year
Surface Rejuvenation... 9th year
Surface Seal.. 12th year
Rejuvenate the surface.. 15th year
Surface Seal.. 18th year
Structural Overlay and Surface Seal........................ 23rd year
Surface Rejuvenation... 26th year
Surface Seal.. 29th year
Surface Rejuvenation... 32nd year
Surface Seal.. 35th year
New Pavement Structure... 40th year

Category Number 5: **Gravel Roads**

The typical type of treatment and time intervals for this category include:

Treatment	Time Interval - Years
Gravel Replacement	5

FLEXIBLE PAVEMENT SURFACE TREATMENT

NON STRUCTURAL/THIN BITUMINOUS OVERLAY

Description: A dense graded bituminous mixture limited to a 1 ½ inch thickness.

Purpose: A non structural bituminous overlay is the highest type of fix available in the Preventative Maintenance Program. It will protect the pavement structure, slow the rate of pavement deterioration, correct many pavement surface deficiencies, improve the ride quality and add strength to the existing pavement structure.

Existing pavement condition: The existing pavement should exhibit a good base condition and a uniform cross section. The visible surface distress may include; moderate to severe raveling, longitudinal and transverse cracks with the first signs of slight raveling and secondary cracking, first signs of longitudinal cracks near the pavement edge, block cracking up to 50 % of the surface area, extensive to severe flushing or polishing and/or some patching or edge wedging in good condition. The pavement may also have some minor base failures and depressions.

Existing pavement surface preparation: This preparation work should be limited to the repair of the minor base failures and depressions, the filling of voids in the pavement surface, the removal of any patched area with a very high asphalt content that may bleed up through the new bituminous surface, the correction of severely tented joints and the correction of deficient superelevations, if required.

Performance: This fix is preferred for higher traffic volume roads. It performs best on full depth flexible pavement structures, but is also applicable to composite pavements. Over a full depth flexible pavement structure, the design life should exceed seven years. On a composite pavement structure, the design life could be less, depending on the extent of the reflective cracking.

Limitations: A non structural bituminous overlay should not be placed on a Plant Mix Seal Coat. It should also not be placed on the following existing pavement conditions: severely distressed concrete pavement, rutted bituminous pavement, pavement with a weak base, or a delaminated bituminous surface. Coring the existing pavement helps with evaluating its condition.

ROTOMILLING

Description: The removal of an existing bituminous surface by the cold milling method. Rotomilling is usually followed by an non structural bituminous overlay.

Purpose: In the Preventative Maintenance Program, the cold milling operation is used to remove an old plant mix seal coat layer or layers. The cold milling operation is also used to correct rutting in the existing bituminous surface layer where the rutting is not caused by a weak base and when the condition of the existing pavement has deteriorated to a point where it is not practical to correct the rutting problem by a more economical treatment. It is also used to remove an existing bituminous course that is delaminating. Existing pavement crown and superelevation sections that have been identified as having a bad accident history can be corrected by cold milling. Often, only a portion of the existing cross section needs a preventative maintenance fix. In these cases, it is more economical to remove the existing bituminous surface in that lane by cold milling and do nothing or do a less expensive fix on the less deteriorated portions of the cross section. In a curb and gutter section, cold milling can be used to remove a portion of the existing bituminous surface to retain the existing curb face. Cold milling can also be used in those areas where the existing pavement grade cannot be raised.

Existing pavement condition: Cold milling should not expose a weak pavement layer for the non structural bituminous overlay that is usually applied afterwards. Coring the existing pavement should be done to make sure that this does not happen.

Existing pavement surface preparation: None.

Performance: NA

Limitations: The limitations are the same as described under “Existing pavement condition”.

FLEXIBLE PAVEMENT SURFACE TREATMENT

CHIP SEAL

Description: A chip seal is the application of a polymer modified asphalt emulsion with a cover aggregate. A single or a double chip seal can be used in the Preventative Maintenance Program.

Purpose: A chip seal will rejuvenate or retard the oxidation of the asphalt at the existing pavement surface, improve skid resistance of the pavement surface, seal fine surface cracks in the pavement thus reducing the intrusion of water into the pavement structure, and will retard the raveling of aggregate from a weathered pavement surface.

Existing pavement condition: The existing pavement should exhibit a good cross section and a good base. The visible surface distress may include slight raveling and surface wear, longitudinal and transverse cracks with a minor amount of secondary cracking and slight raveling along the crack face, first signs of block cracking, slight to moderate flushing or polishing and/or an occasional patch in good condition.

Existing pavement surface preparation: For single chip seals all visible cracks and construction joints should be sealed. On double chip seals all cracks and construction joints greater than twelve inches in length and greater than one fourth of an inch in width should be sealed. When the number of cracks and construction joints to be sealed reach a certain quantity, it may be more economical and practical to place a double chip seal in lieu of a single chip seal and eliminate the sealing of any cracks and construction joints.

Performance: The life expectancy of a polymer modified single chip seal is approximately five to seven years. Double chip seals are reported to give about twice the service life as a single chip seal. This is with the assumption that both type of applications were placed on pavements in like condition. Since the double chip seals in the Preventative Program are used in sealing the cracks and construction joints in the existing pavement in lieu of crack sealing, the life expectancy may not be twice that of a single chip seal.

Limitations: Chip seals are used only on lower volume roads and on bituminous surfaced shoulders. Chip seals should not be placed on a plant mix seal coat. The construction season for this work is relatively short. Chip seals should not be placed in cool weather. It usually requires about one month of warm weather following construction for the aggregate particles to become reoriented and properly embedded in the asphalt membrane. Windshield damage is another problem. Loose aggregate not embedded in the asphalt membrane will become airborne and possibly damage windshields of vehicles of the traveling public. Traffic noise will also increase after a chip seal is placed.

FLEXIBLE PAVEMENT SURFACE TREATMENT

OPEN GRADED/PLANT MIX SEAL COAT

Description: A Plant Mix Seal Coat (PMSC) is an open or gap graded bituminous mixture limited to one inch of thickness.

Purpose: A PMSC is used to seal the pavement surface, improve ride quality and skid resistance.

Existing pavement condition: The existing pavement must be structurally sound. There should not be any rutting. The existing pavement should exhibit a uniform cross section and a good base. The visible surface distress may include slight to moderate cracking, minor surface irregularities, flushed or polished surface and/or severe raveling.

Existing pavement surface preparation: If the pavement is rough, milling or adding a leveling course is recommended. Any ruts in the surface should be milled or leveled out with a leveling course so that water will not pond in the ruts, retain moisture in the layer and encourage stripping. Existing cracks should be filled flush with the pavement surface.

Performance: The life expectancy of a PMSC is approximately eight to twelve years.

Limitations: No more than two PMSC layers can be placed before the PMSC needs to be removed. It is unstable due to the gap grading and is also susceptible to stripping if it is overlaid or chip sealed. Crack sealing and patching is a problem. Sealed cracks and patches at the edge of the pavement can create a dam in the surface that inhibits the flow of water. As a result, the surface can become saturated, which can lead to stripping. Deicing salt requirements are increased because the salt penetrates into the pavement surface. Freezing of moisture in the pavement surface can develop excessive stresses and promote fracture. Over time, the drainage ability may decrease due to the clogging of the voids with dust and the application of winter deicing chemicals and abrasives.

FLEXIBLE PAVEMENT SURFACE TREATMENT

SLURRY SEAL

Description: Slurry seal is a mixture of asphalt emulsion, well-graded fine aggregate (sand) and mineral filler (in most cases) mixed with water to produce slurry consistency. Additives such as portland cement, hydrated lime, or aluminum sulfate liquids are often used to aid setting the slurry.

Purpose: Slurry seal will seal minor surface cracks and voids, retard surface raveling, improve surface friction characteristics, and delineate different pavement surface areas.

Existing pavement condition: The existing pavement should not have large cracks that displace under traffic. The existing pavement has to be stable with no excessive rutting or shoving.

Existing pavement surface preparation: Cracked sealing and patching must be done enough in advance of the slurry seal to allow for complete curing. Immediately prior to construction of the slurry seal, the pavement should be cleaned with a power broom. A tack coat should be used on dry or raveled pavements.

Performance: The life expectancy of a slurry seal is approximately three to five years.

Limitations: Slurry seal should not be applied on a pavement where the cracks move under traffic. Slurry seals require longer curing time than chip seals. It should not be placed during rain or if rain is expected before the slurry is set.

FLEXIBLE PAVEMENT SURFACE TREATMENT

MICRO-SURFACING

Description: Micro-Surfacing is a mixture of polymer modified asphalt emulsion, mineral aggregate, mineral filler, water, and other additives, properly proportioned, mixed, and spread on a paved surface.

Purpose: Micro-Surfacing has been used to correct certain pavement surface deficiencies including rutting, minor surface irregularities, slippery surfaces and raveling.

Existing pavement condition: The existing pavement should exhibit a uniform cross section and a good base. The visible surface distress may include slight to moderate cracking, rutting, minor surface irregularities, flushed or polished surface and/or severe raveling. The pavement should not have large cracks that displace under traffic.

Existing pavement surface preparation: Crack filling and pothole repairs must be done far enough in advance to allow for complete curing.

Performance: A Micro-Surface performs well on high volume roadways to correct the pavement surface conditions described above. The life expectancy for this surface treatment should exceed five years.

Limitations: Micro-Surfacing should not be used on a pavement with moderate to heavy surface cracks. Due to it's brittle nature, it is a poor crack sealer. Micro-Surfacing mixes are very aggregate specific because of the chemically triggered, quick reaction characteristics of the mixture. Micro-Surfacing requires special application equipment. The contractor has to have good experience.

FLEXIBLE PAVEMENT SURFACE TREATMENT

REJUVENATION

Description: Rejuvenation is a light application of diluted asphalt emulsion without an aggregate cover.

Purpose: The purpose of rejuvenation is to renew old asphalt surfaces, seal small cracks and surface voids, address raveling of chip seals and open graded surfaces on high volume roads, and to maintain and delineate shoulders on high volume roads. Higher application rates should be used for passing lanes and shoulders where the traffic is lighter than on travel lanes where the traffic is heavier.

Existing pavement condition: The existing pavement surface must be sufficiently porous to absorb a substantial amount of the emulsion. The pavement should display low to moderate weathering or raveling.

Existing pavement surface preparation: The pavement needs to be thoroughly cleaned before applying the rejuvenator. The pavement should also be dry. A small area of the pavement should be tested with different application rates to determine the optimal application rate for the entire area to be rejuvenated.

Performance: The life of the treatment depends on the condition of the pavement when the treatment was placed, the amount applied, traffic, and the environmental conditions.

Limitations: Rejuvenation should never be done on a pavement that has low skid resistance or where the asphalt is unstable as indicated by rutting or shoving. Rejuvenation should be applied very cautiously to interstate and high volume roads is applied at all. These roads might not need rejuvenation in the travel lanes because of the heavy traffic. Rejuvenation of the shoulders on these roads is a good use of rejuvenators.

FLEXIBLE PAVEMENT SURFACE TREATMENT

LANE LEVELING

Description: A "lane leveling" is an application of a course of bituminous mixed materials.

Purpose: The purpose of lane "leveling" is to restore profile and cross section to the pavement. It is sometimes followed by an overlay.

Existing pavement condition: The existing pavement has a deficient profile and cross section.

Existing pavement surface preparation: The existing pavement should be swept clean before applying a "lane leveling."

Performance: A "lane leveling" will usually not last very long if no overlay is placed over it. The mix is usually a finer gradation mix and is sometimes placed with a grader/maintainer in thin lifts, so the quality of pavement is not as good as the quality of a coarser gradation mix placed with a laydown machine. The performance is very dependent on the mix, how well is compacted, the condition of the existing pavement, and the expected traffic.

Limitations: Lane leveling is not a good application for the interstate because of the heavy traffic. Lane leveling is not a good application for all rutting. Where the rutting is from a stripped pavement, lane leveling will sometimes accelerate the stripping. The cause of the rutting needs to be determined before lane leveling is applied.

CRACK SEALING

Description: Crack sealing consists of cleaning the crack in the bituminous pavement surface and placing the specified materials into the crack to substantially reduce infiltration of water and to reinforce the adjacent pavement.

Purpose: The purpose of crack sealing the surface of the bituminous pavement is to prevent water and incompressible from entering the pavement structure. This treatment is mostly used as a surface preparation for surface seal treatments. For its use, cracks are filled by overbanding. It is rarely used as a stand alone Preventative Maintenance treatment due to excess wear or failure.

Existing pavement condition: The condition of the existing bituminous surface depends upon the other Preventative Maintenance treatment the surface seal treatment will be combined with.

Existing pavement surface preparation: Before sealing, the crack must be completely free of dirt, dust, and other materials that might prevent bonding of the sealant.

Performance: This treatment should help extend the service life of the treatment it is being used with and thus extend the service life of the pavement structure.

Limitations: If this treatment is to be used as a stand alone, caution should be taken in the selection of a pavement where there are too many cracks.

RIGID PAVEMENT TREATMENT

CONCRETE JOINT RESEALING AND JOINT SPALL REPAIR

Description: This work includes the removal of the existing joint seals, the repair of the pavement spalls at the joint to restore the shape of the joint, and the resealing of the joint with the specified material.

Purpose: The purpose of resealing the concrete pavement joints is to prevent water and incompressible from entering the pavement structure, thus slowing the rate of deterioration of the concrete pavement.

Existing pavement condition: The existing joint sealant should be deteriorated or missing.

Existing pavement surface preparation: Before resealing, the old sealant must be removed. At the time of sealing, the crack must be completely free of dirt, dust, and other materials that might prevent bonding of the sealant.

Performance: A properly placed concrete pavement seal should have a service life of approximately ten years and should slow the deterioration rate of the concrete pavement.

Limitations: If the existing pavement is badly deteriorated, sealing joints may not be effective.

RIGID PAVEMENT TREATMENT

CONCRETE CRACK SEALING

Description: Crack sealing involves the sawing or routing, cleaning and sealing of cracks in the concrete pavement that are greater than 12 inches in length and greater than 1/8 inch in width. If the crack is greater than ½ inch in width a backer rod must be used.

Purpose: The purpose of sealing the cracks in the concrete pavement is to reduce the water and incompressible from entering the pavement structure and thus slowing the deterioration rate of the pavement.

Existing pavement condition: The sealing of the cracks in the concrete pavement should begin on a pavement that is ten to fifteen years old. Subsequent Preventative Maintenance crack sealing projects should follow every five years or until the condition of the pavement requires extensive work that is beyond the scope of the Preventative Maintenance Program.

Existing pavement surface preparation: Before sealing, the crack must be completely free of dirt, dust, and other materials that might prevent bonding of the sealant.

Performance: Crack sealing should help slow the deterioration rate of the concrete pavement. This treatment is best used in conjunction with other treatments of rigid pavements such as joint resealing and minor spall repair and /or full depth concrete joint repair.

Limitations: If the existing pavement is badly deteriorated, sealing cracks may not be effective.

RIGID PAVEMENT TREATMENT

DIAMOND GRINDING

Description: This work consists of diamond grinding the entire lane width as specified on the plans.

Purpose: Diamond grinding is used to restore the surface profile and cross section of a concrete pavement and provide an improvement to the ride quality. Benefits from diamond grinding include: the removal of joint and crack faults, the removal of wheel ruts caused by studded tires, the removal of slab warping at joints, the restoration of transverse drainage, and the improvement of skid resistance. If appropriate, only one lane of a multi-lane roadway can be improved by diamond grinding.

Existing pavement condition: The existing pavement should exhibit a uniform cross section and a good base. The visible surface distress may include joint and crack faults not exceeding 1/4 inch, rut depths less than 1/4 inch, minor slab warping at the joints, and moderate to severe polishing or scaling not over twenty five percent of the surface area.

Existing pavement surface preparation: Diamond grinding should not be viewed as a one step solution to treating the concrete pavement surface. Full depth concrete repairs, partial depth spall repairs, sealing of all the joints and cracks, slab undersealing and jacking, and dowel bar retrofit should be included along with diamond grinding in a complete concrete pavement treatment package project. Diamond grinding should be considered when the average international ride index (IRI) is greater than 140, average friction is 30 or less or there are 25 or more full depth repairs per mile.

Existing pavement surface preparation: None

Performance: Faulting at the joints and cracks may return after several years of service to the condition prior to diamond grinding. This will depend on several factors, including the joint efficiency of the pavement and the amount and quality of concurrent concrete pavement treatment work. The improved skid resistance due to diamond grinding depends on the final micro texture and macro texture and the hardness and polishing characteristics of the aggregates. The improved skid values will decline until they reach the skid levels of the original surface at which point the values will generally remain steady.

Limitations: Diamond grinding should generally not be used on concrete pavements where the faulting is greater than 1/4 inch. Greater fault depths will greatly increase the unit cost of diamond grinding. As mentioned above, diamond grinding should not be used as a one step solution to treating the deficiencies of the concrete pavement.

RIGID PAVEMENT TREATMENT

SLAB JACKING AND/OR UNDERSEALING

Description: Undersealing is filling the voids under the concrete slab with a material such as cement grout or high density polyurethane under pressure through holes drilled in the slab. Slab jacking is overfilling the voids under the concrete slab to lift the slab.

Purpose: The purpose of filling the voids under a slab is to reestablish a stable base to support the slab to prevent the slab from faulting and cracking. It is also done to prevent the material under the slab from pumping out from under the slab when moisture is present. The purpose of slab jacking is to restore the pavement cross section and/or profile.

Existing pavement condition: There could be excessive vertical movement of the slab at joints or cracks. Pumping can be identified by staining of the concrete surface. The existing slabs will be faulted and there may be cracking of the slabs were there is little support.

Existing pavement surface preparation: None

Performance: Undersealing provides continuous support under the slab which in turn increases the structural integrity of the pavement section and extends its performance life. Since undersealing or slab jacking does not restore joint transfer efficiency, it is usually done as part of a restoration process that can include grinding and retrofit of dowel bars. Such restoration projects can extend the life of pavements by 10 years.

Limitations: Undersealing and slab jacking requires considerable expertise, trained personnel, and special equipment. Undersealing and slab jacking is quite often ineffective when used alone as faulting quickly returns. It should be used with load transfer restoration and diamond grinding to restore ride quality.

Section 3F
Alternative Selection

Final pavement design alternative selection will be made with the Regional and state pavement Management Engineers based on any combination of the following:

- Life-Cycle Cost Analysis Results
- Construct ability
- Feasibility
- Availability of Materials
- Future Rehabilitation Options
- Traffic Volume Considerations (during construction & long-term)
- Potential Climate Problems
- Performance of Similar Pavements in the Area
- Future Maintenance Requirements
- User Costs
- Traffic Control Considerations
- Worker Safety During Construction
- Other Safety Considerations
- Overall Pavement Mange ment Consideration
- Engineering Judgement
- Other Considerations

Section 3G

New Pavement Design Report Format

I. Site Conditions.

Report soils information.
List the design CBR value.
List the frost penetration depth.

II. Traffic.

List expected traffic (design ESALs) from Project Traffic Information sheets if available; otherwise, estimate the expected traffic (design ESALs) from AADT data or manual traffic counts.

III. Develop Feasible Alternatives.

List alternatives and costs.

IV. Conduct An Engineering Analysis For Each Feasible Alternative.

Do a life cycle cost analysis.

V. Select The Appropriate Alternative.

Attachments:

soils information (design CBR calculations as a minimum)
frost penetration map
ESAL calculations
pavement design calculations for each alternative
initial cost estimates for each alternative
life cycle cost analysis
high and low temperature maps for PG asphalt